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THE Association was organized in Boston, Mass., on June 21, 1882, with the object of providing its members with means of social intercourse and for the exchange of knowledge pertaining to the construction and management of water works. From an original membership of only TWENTY-SEVEN, its growth has prospered until now it includes the names of 800 men. Its membership is divided into six classes, viz.:

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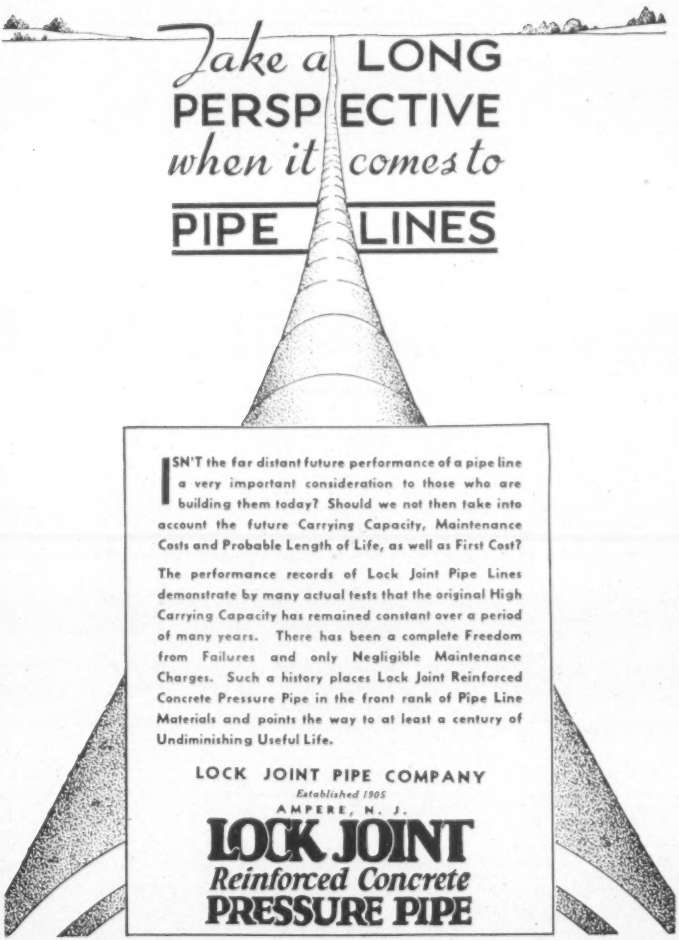

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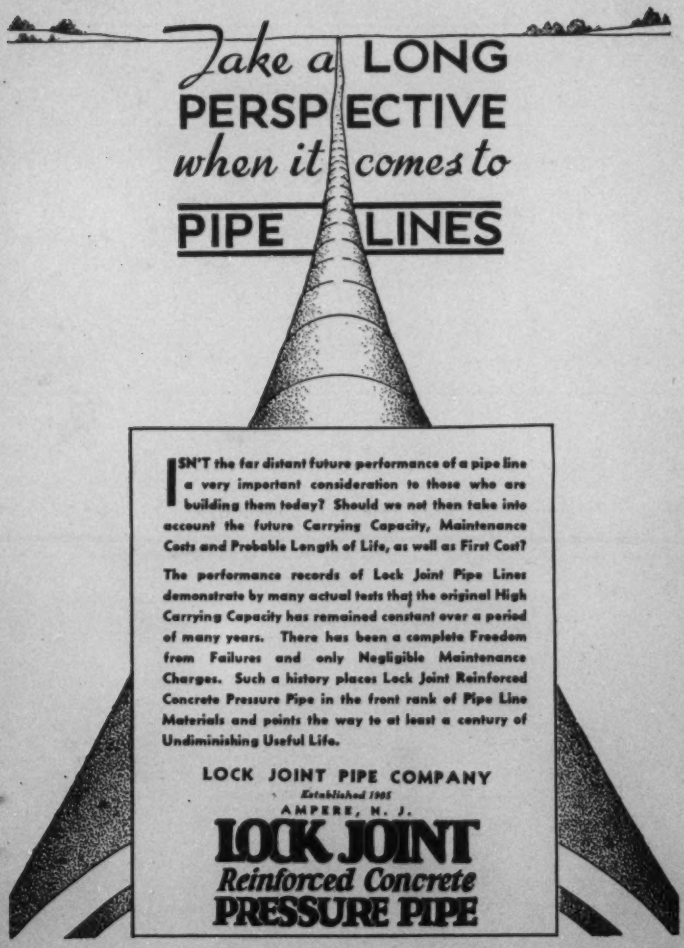
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New England Water Works Association

ORGANIZED 1882.

Vol. XLVIII. June, 1934. No. 2.

This Association, as a body, is not responsible for the statements or opinions of any of its members.

THE COBBLE MOUNTAIN POWER TUNNEL.

BY HARRY H. HATCH.*

[*Received January 29, 1934.*]

The Cobble Mountain development is unique in many ways. It is a combined water-supply and water-power project, costing about six million dollars, being built by a municipality. The watershed tributary to the project, is only 46 sq. miles yielding an average flow of about 90 c.f.s. Yet there is included the construction of the highest hydraulic fill dam in the world. The fall of head is 334 ft. minimum and 456 ft. maximum, with working average of about 420 ft. The power house and the power tunnel have a capacity of at least ten times the mean flow of the stream.

The city of Springfield has been able to sell the power at an annual figure of $270 000 for the next 30 years, without jeopardizing the full benefit of the ultimate development of the watershed for its municipal water supply purposes.

The Cobble Mountain project has been a happy combination both for the city of Springfield and the Turners Falls Power & Electric Company. The city would have been unable at this time to develop its watershed to full capacity without the financial returns through the power company; and the latter has acquired the most important water power within its territory to supply its badly needed requirements without any financial obligations other than the yearly payment of rental.

The Cobble Mountain tunnel (Fig. 1) is built especially for the development of water power. Its entrance is located 1200 ft. south of the spillway of Cobble Mountain Reservoir and follows the northern slope of Sweetman Mountain in an easterly direction to a point south of and almost directly above the power house. The tunnel will always be under pressure, and its course was laid to give at least 150 ft. cover on top, except near the end where a steel lining has been provided.

The present average daily water consumption of the city is about 28 c.f.s. The power company will draw this amount, or whatever the city requires for its daily use, every day during the peak-load hours. The city has more than one day's supply capacity in reservoirs or basins below the Power

*Engineer in charge Cobble Mountain Reservoir, Springfield Water Works, Springfield, Mass.

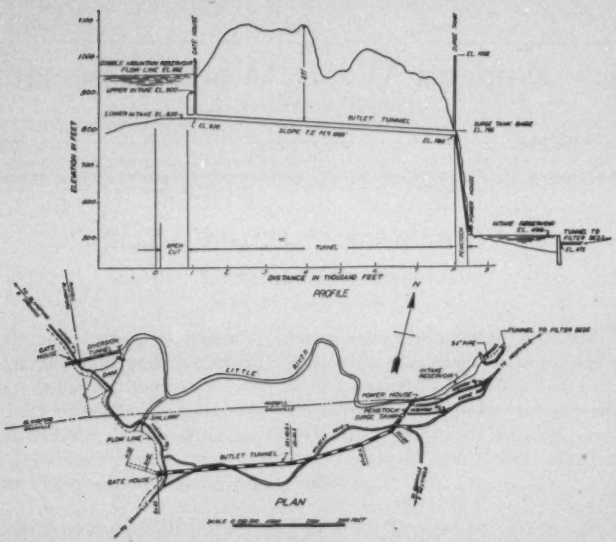

FIG. 1.—COBBLE MOUNTAIN POWER TUNNEL.
Location, plan, and profile.

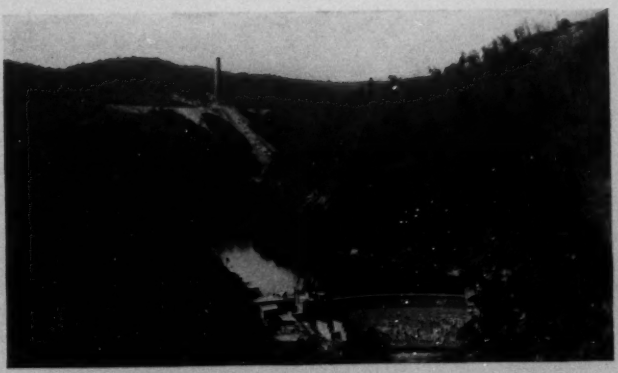

FIG. 2.—POWER HOUSE DIRECTLY BELOW THE SURGE TANK.
Intake Reservoir of the Springfield Water Supply in foreground.

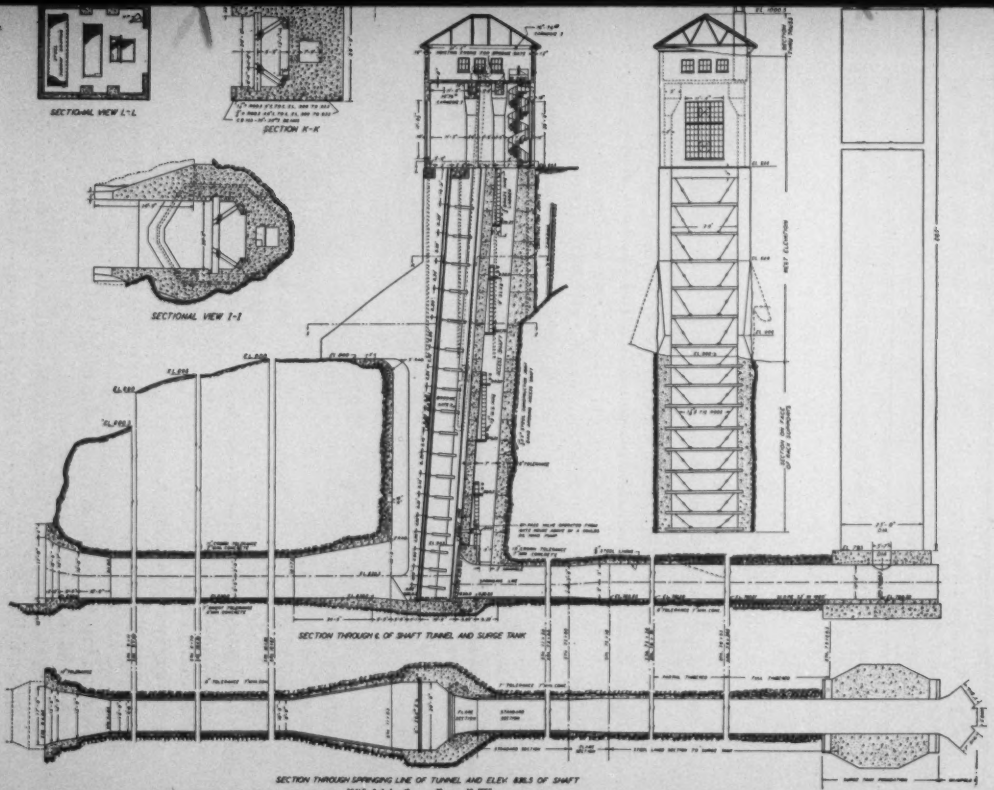
SECTIONAL VIEW L-L
SECTION K-K
SECTIONAL VIEW I-I
SECTION THROUGH ℄ OF SHAFT TUNNEL AND SURGE TANK
SECTION THROUGH SPRINGING LINE OF TUNNEL AND ELEV. BRLS OF SHAFT

FIG. 3.—COBBLE MOUNTAIN POWER TUNNEL. Typical sections of tunnel, shaft, gate house and surge tank.

House (Fig. 2), and the daily supply can be drawn any time during the 24 hours. The difference between the mean flow of the stream and the city's daily draft will be impounded in Cobble Mountain Reservoir for use, under certain restrictions, at the convenience of the power company. As the tunnel and the power plant capacities are ten times that of the mean flow of the river, the impounded yearly excess water can be drawn during a period as short as about five weeks. The power company will use this water during dry seasons when heretofore it had to operate steam plants to supplement its hydraulic plants.

Description of Tunnel, Shaft and Gate House.

The tunnel has two inlets with level grades, the lower at elevation 830 ft. and the upper at elevation 900 ft. (Fig. 3).

The approach channel of the lower inlet is through solid rock; length 450 ft., bottom width 15 ft., and side slopes 4 on 1. The lower inlet is circular in shape, 17 ft. in diameter, and is reduced to a 10 ft. by 11 ft.-6 in. section within a distance of 20 ft. This section is 168 ft. long and has 100 sq. ft. of area. It has a level grade with invert elevation at 830 ft. There follows a 30-ft. transition section, the end of which at the control shaft is 18 ft.-8 in. by 16 ft.-6 in. The portal is so arranged that it can be closed by an emergency hollow steel gate for the purpose of repairing the tunnel.

The upper inlet is 20 ft. wide at the bottom with a slope of 2 to 1 for dirt, and ¼ to 1 for rock excavation. Near the shaft there are two retaining walls as part of the concrete structure of the main shaft. These walls are 30 ft. long, 6 ft. high at the end, rising to a height of 28 ft. at the shaft.

The main shaft has an octagonal shape with about 390 sq. ft. of area from elevation 830 to 900 (Fig. 4). The easterly side, on which the gate will be operated, has a slope of 15.8 on 1. This side forms the 3-ft. thick partition wall between the main shaft and the 6 ft. by 7 ft. small shaft. The main shaft has 18 rack supports spaced, center to center, from 4.95 ft. at the bottom to 8.25 ft. at the top. These rack supports are made of 24-in., 70-lb. I beams with two 14-in., 58-lb. I beam horizontal knee braces, and 1⅛-in. diameter vertically diagonal tie rods. The trash racks which are bolted to these rack supports are made of ⅜-in. by 5-in. bars spaced 3⅜ in. center to center and extend from the bottom to the top of the shaft.

The small shaft is built to serve as an air vent and to give access to the tunnel. It has six landings and ladders made of ¾-in. diameter brass pipe rungs cast into the concrete. At its bottom, right at the crown of the tunnel, there is a 36-in. diameter manhole leading to the tunnel.

The gate house, built directly over the shaft, is a two story, reinforced concrete building, 28 ft. by 32 ft. in plan, and 45 ft. high. In it is installed the equipment for operating the Broome self-closing sluice gate, which is 12 ft. wide by 13 ft. high, weighing about 15 tons, and operated by a Philips & Davies 200-ton capacity electric-driven double drum hoist.

Near the manhole, in the access shaft there is a 12-in. bypass valve operated by a Goulds oil-pressure hand-pump in the gate house. With the Broome gate closed and the tunnel empty, this valve permits filling the tunnel with water from the power house to the shaft, thus equalizing the pressure on both sides of the gate so that it can be raised easily.

The area of the tunnel at the gate seat is 156 sq. ft. It is brought down within a distance of 6.7 ft. to a horse-shoe section which is 10 ft. by 9 ft.-4 in., has an area of 78.96 sq. ft. and is 6 376 ft. long. There follows a transition section of about 12 ft., changing the horse shoe into a 10-ft. diameter circle. In its terminal length of about 450 ft. the tunnel has a 7/16-in. thick

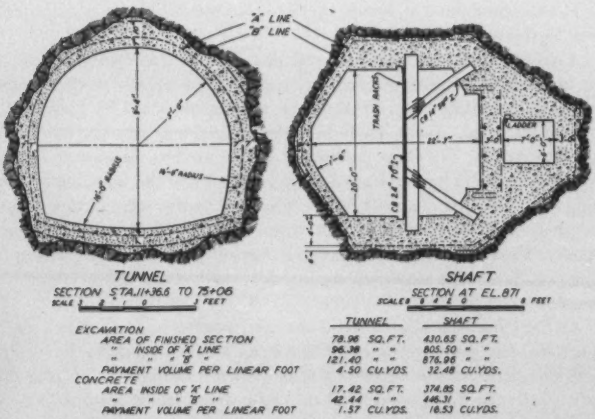
"A" LINE
"B" LINE
TRASH RACKS
CABLE
TUNNEL
SECTION STA.11+36.6 TO 75+06
SCALE 1 0 1 2 FEET
SHAFT
SECTION AT EL. 871
SCALE 0 4 8 12 FEET
TUNNEL SHAFT
EXCAVATION
AREA OF FINISHED SECTION 78.96 SQ.FT. 430.65 SQ.FT.
" INSIDE OF "A" LINE 96.38 " " 805.50 " "
" " "B" " 121.40 " " 876.96 " "
PAYMENT VOLUME PER LINEAR FOOT 4.50 CU.YDS. 32.48 CU.YDS.
CONCRETE
AREA INSIDE OF "A" LINE 17.42 SQ.FT. 374.85 SQ.FT.
" " "B" " 42.44 " " 446.31 " "
PAYMENT VOLUME PER LINEAR FOOT 1.57 CU.YDS. 16.53 CU.YDS.

FIG. 4.— TYPICAL CROSS-SECTIONS OF TUNNEL AND SHAFT.

and 10-ft. diameter steel lining, all joints welded, extending through the surge tank foundation. From the shaft to the surge tank there is a uniform slope of 0.72 per cent., and there occur three horizontal angles in the alignment. The total length of the tunnel is 7 100 ft.

The surge tank is 25 ft. in diameter and 207 ft. high. Its top is 50 ft. above the maximum flow-line of the reservoir. The tank is made up of riveted steel plates, varying from 1-in. thickness at the bottom to 7/16 in. at the top. The bottom of the surge tank is 5 ft. above the top of the steel lining of the tunnel. The two are connected vertically by a 5 ft.-7 in. diameter steel pipe. Near the surge tank foundation, the steel lining is connected into a manifold with three outlets, the middle one of 5-ft. diameter and the other two of 7-ft. diameter, leading into 3 penstock lines. The middle penstock is 5 ft.-6 in. in diameter with thickness of plate varying from 3/8 to 5/8 in. and the other two 7 ft.-9 in. in diameter with plates from

$\frac{3}{8}$ to $\frac{7}{8}$ in. thick. The penstocks are about 660 ft. long with a 284 ft. drop in elevation. The power units consist of two 13 620 and one 5 760 k.v.a. generators.

Control of the Tunnel Lines and Grades.

The ends of the tunnel were located on the ground with the help of topographic maps, and careful surveys with tape and transit were made on the surface over the possible tunnel courses. The traverse was checked independently; the resulting error of closure was 1 in 15 000. Later on, the survey was also checked by a triangulation system, the difference being only 0.6 ft. in a distance of about 8 000. The few hundred dollars spent for the triangulation system was well worth the peace of mind that it produced.

Independent lines of levels, to the nearest 0.001 ft. were run from one portal to the other and back.

A triangulation station was established over the tunnel course near each heading, and permanent bench marks and targets were established far enough away from the portal so as not be to disturbed by blasting.

East Heading. About 4 500 ft. of the tunnel were driven from the east, or outlet, portal. The three horizontal angles in the tunnel were all within this distance. The line and grade were carried into the heading by precise taping for a distance of about 500 ft. from the nearby triangulation station, making use also of the permanent target and the bench mark set for this purpose. A hub was established on the tunnel center line a few feet in front of the portal. Its station was carefully determined and the necessary reference points put in.

As the excavation assumed a normal progress in the tunnel, the lines and grades were given to the heading boss at least once a day. Every 100 ft. a hole was driven into the crown of the tunnel and a wooden plug driven into it. Into the plugs were driven spads, from which plumb bobs were suspended for transit points. The spads were used also for turning points and bench marks. Ordinary transits and tapes were used for every day work. Line was carried ahead always by double centering. Before any angle in this drift was turned, the course was measured at least twice by precise taping. The precise work was done with just as little interference to the progress of construction as possible. For this reason the engineering party sometimes had as many as nine men. The equipment for precise work consisted of:

One Buff & Buff tunnel transit, with special attachments.
One 200-ft. steel standardized tape.
One 50-lb. spring balance.
Two 5-ft. long steel bars.
One Wider Thermometer, and extension tripod, level rod, plumb bob, flash lights, etc.

Corrections were made for absolute length, sag, temperature, and difference in elevation of the ends of the tape. The tape was standardized with a 35-lb. pull. The same pull was used in the taping so as to eliminate the correction for it.

The exterior angles were so small that the points of intersection remained within the tunnel section. At each angle point the angles were turned every day with moderate care until the heading advanced a few hundred feet to establish a good control in the new course. Then a set of 12 interior and 12 exterior angles was turned, and the mean taken, using just as long a back-sight as possible. This work was checked independently and the necessary adjustment was made in the next course.

West Heading. The contractor sank the shaft and started the easterly drift before boring through the section to the west, or inlet, portal. Both

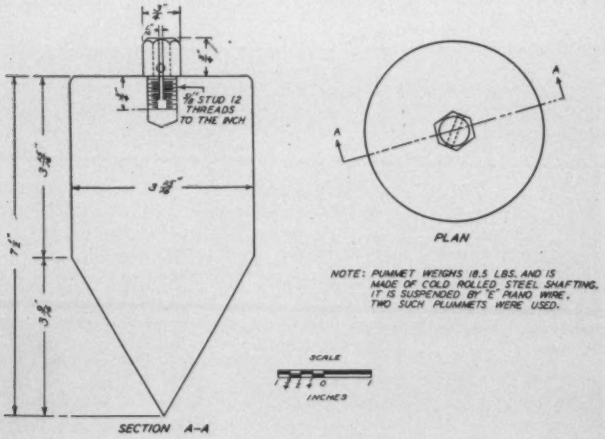
⅝" STUD 12 THREADS TO THE INCH
7'
3 3/8"
3 3/8"
SECTION A-A
A
A
PLAN
NOTE: PLUMMET WEIGHS 18.5 LBS. AND IS MADE OF COLD ROLLED STEEL SHAFTING. IT IS SUSPENDED BY "E" PIANO WIRE. TWO SUCH PLUMMETS WERE USED.
SCALE
INCHES

FIG. 5.—SHAFT PLUMMET.

the alignment and grades had to be carried down the shaft into this heading.

The elevation was carried to a sharp-edged board fastened on a heavy cross timber over the shaft. From this point a tape was suspended with a weight at its lower end to make it taut. With the level set at the bottom of the shaft, the control for the grade was established.

Two 18-lb. plummets (Fig. 5), were suspended with piano wires from the top into buckets filled with heavy oil at the bottom of the shaft. By trial the transit was set on line and the alignment carried into the heading. The transfer of the lines and grade down the shaft into the heading was done often enough and checked independently until satisfactory control was established. About 2 400 ft. of tunnel were driven with this control. The taping and leveling in the tunnel were carried to 0.001 ft. The error in the meeting of the headings was only 0.1 ft. in both alignment and grade.

For trimming the tunnel and also for the control of pouring the invert,

side walls and the arch, a plug was set in on each side of the springing line every 25 ft., in addition to the plugs set into the crown every 100 ft. during the excavation. Into these plugs were driven nails with their heads exactly on the minimum concrete line and their centers on the springing line for that particular station.

Shaft and Tunnel Excavation.

The shaft was sunk through solid rock, except for about 14 ft. of earth cover at the top. About 24 sump holes and 84 other down holes were put in from 6 to 8 ft. deep, using 4 jackhamers. The position of the sump holes

Fig. 6.—East Portal Timber Protection.

was shifted from one side to the other. About 2 lb. of 40-per cent. DuPont special gelatin dynamite was used per cubic yard of rock excavation. The sump holes were shot direct and the others with up to 5 delays. Muck was loaded by hand into 1-yd. steel buckets and hoisted up to the surface by a 15-ton Northwest crane. The shaft had about 31 cu. yd. excavation per foot, and the general average progress was 1.3 ft. per working day.

Well outside of the concrete section of the shaft, permanent timber cribbing 12-in. round, was put in near the surface through the earth section of the shaft. When the shaft was completed a head frame and guides were built and a one-car cage installed for hoisting the muck from the tunnel to the surface.

At the entrance of the east portal, the drift was made through soft ground for about 90 ft. with timber sheathing all around (Fig. 6). From here on the entire excavation was through solid rock, mica schist with inter-

mittent narrow veins of gneiss. Occasionally, however, veins of Williamsburg granite were encountered. Mica schist and gneiss were easy to drill, but shooting was very hard. The cut holes sometimes were shot up to 3 times. Williamsburg granite was very hard to drill, sometimes a bit drilling not more than two feet, but it shot very easily.

The entire tunnel was driven with full face heading, except occasionally. When the mucking of the previous shot was delayed, heading and bench methods were used for short distances. As soon as a shot was made, a cross

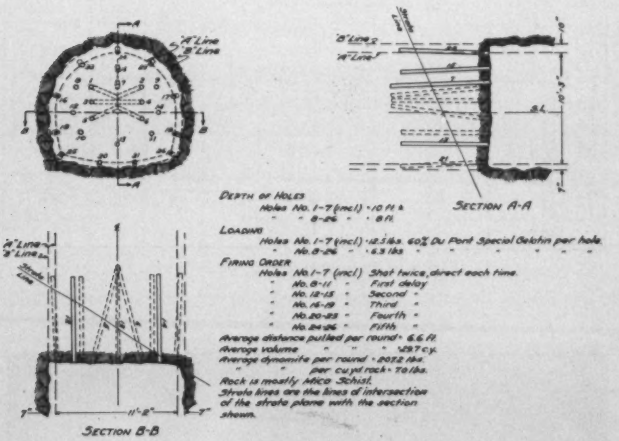
Depth of Holes
Holes No. 1-7 (incl) - 10 ft.
" " 8-26 " - 8 ft.
Loading
Holes No. 1-7 (incl) - 12.5 lbs. 60% Du Pont Special Gelatin per hole.
" No. 8-26 " - 6.3 lbs. " " " " "
Firing Order
Holes No. 1-7 (incl) Shot twice, direct each time.
" No. 8-11 " First delay
" No. 12-15 " Second "
" No. 16-19 " Third "
" No. 20-23 " Fourth "
" No. 24-26 " Fifth "
Average distance pulled per round - 6.6 ft.
Average volume " " " - 427 cu.
Average dynamite per round - 20.2 lbs.
" " per cu. yd rock - 7 lbs.
Rock is mostly Mica Schist.
Strata lines are the lines of intersection of the strata plane with the section shown.
Section A-A
Section B-B

Fig. 7.— Method of Drilling and Shooting Heading of Tunnel.

bar was set up and 3 Leyners were mounted on it. While the mucking was being done, the drilling crew drilled the upper half of the heading standing on the muck pile and the remaining holes were drilled as soon as the mucking was completed. The total number of holes in the heading varied from 25 to 32; most of the tunnel was driven with 26 holes consisting of 7 cut holes, 8 relievers and 11 square ups (Fig. 7). The cut holes were put in as a rule right about the center, but they were moved anywhere depending on the location of the seams in the rock. DuPont 40- and 60-per cent. special gelatine dynamite was used with instantaneous and 5-period delay electric blasting caps. The average draw was about 7 ft. per round, and the average dynamite consumption, exclusive of the tunnel trimming, was about 7 lb. per cu. yd. of excavation. Steady progress was maintained averaging per working day 12.3 ft. at the east and 10.4 ft. at the west heading (Fig. 8). The drilling crew did the loading and shooting.

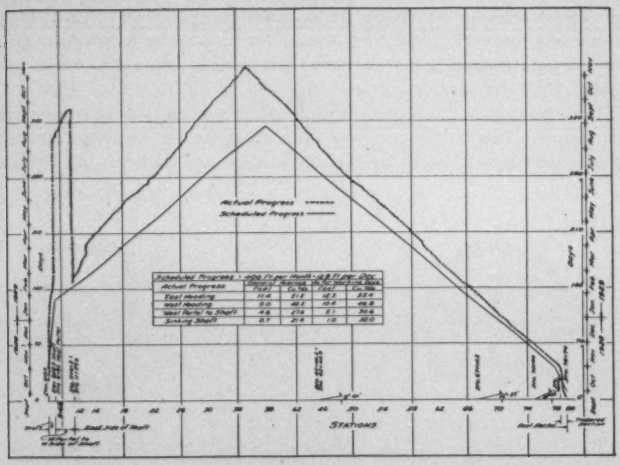

FIG. 8.— PROGRESS CHART OF TUNNEL.

FIG. 9.— MUCKING MACHINE IN TUNNEL.

All dynamite sticks were split and tamped with from 4 to 12 cartridges to the hole; as a rule, when space permitted, as many tamping bags were used in the same hole. Occasionally the hole was filled up with dynamite without any tamping bags.

A small electric-driven Conway mucking machine (Fig. 9) was used at each heading for loading the muck into 1-yd. wooden end-dump cars, which were hauled out in 4- to 6-car trains with an electric dinkey at the east heading over a narrow gauge track, and the muck was used for building a road of about 1 200 ft. outside the portal. The muck at the west heading was hauled by mules, one car at a time, to the bottom of the shaft, where in a 6 ft. by 6 ft. steel cage it was hoisted up to the surface through the shaft and put in spoil banks.

No artificial drainage was necessary at the east heading, as the tunnel here was driven up grade. At the west heading, however, even though the tunnel was comparatively dry, one Worthington, air-driven, 6-in. by 4-in. by 6-in. pump was used to pump the water from the heading into a sump hole at the bottom of the shaft, where an electric-driven Buffalo centrifugal, 3-in. by 2-in. pump was used to pump the water through the shaft to the surface.

The tunnel was ventilated with 4 Coppus Vano 3 200-r.p.m. blowers and with 12-in. diameter canvas ventilating tubing 6 000 ft. long. It took about 10 minutes for ventilation of the tunnel after the cut holes were shot, and 1 hour and 30 minutes after shooting the heading.

There were two 11½-hour shifts per day. The scheduled progress was 400 ft. per month per heading, or about 13 ft. a day. A bonus system was established for the drilling and mucking crews. For every foot extra over the specified distance of tunnel excavation per week the foreman received a bonus of $1.00, the drillers and mucking-machine operator 70 cents and the dummies and muckers 35 cents. At the east portal the specified distance was 98 ft. per 7-day week during the first 2 300 ft. of excavation, 91 ft. during 1 600 ft. of additional excavation and 84 ft. for the remaining 564 ft. of excavation.

Great care was exercised in the location, drilling, and shooting of the square-up holes so as not to have any overbreakage. Considerable time was spent on trimming, with an average cost of 13 cents per square foot area or $5.10 per lineal foot of tunnel. Tunnel excavation per foot was 4.50 cu. yd. at $10 per cu. yd. The trimming cost was more than 13 per cent. of the total excavation. The average section was 1½-in. within the payment line, and it so happened that the value of the excavation and concrete saved per foot was exactly equal to the cost of trimming.

No rock tunnel should be driven too large in the first place so as to avoid trimming later on. Even without the trimming cost, the tunnel would be rather expensive, not only for the extra rock excavation but also for the necessary additional concrete to be put into the overbreaks. There is a limit also as to how tight the bore should be driven in the first place in order

to avoid overbreakage. If a tunnel is driven too tight the trimming expense may amount to more than the value of the anticipated economy in rock excavation and concreting. It is good practice to attempt to remain within the payment lines so that the trimming cost will be offset by the saving in excavation and concrete.

The contractor had 2 Ingersoll-Rand stationary electric air compressors, one of 900 and the other of 600 cu. ft. per min. capacity, near the east portal. This plant supplied the air necessary for the drills, pumps, and the concrete guns. The air to the west heading was supplied by a pipe line from 2 to 8 in. in diameter and about 10 000 ft. long with a 200-cu. ft. steel tank air receiver near the heading. The line was carried over 1½ miles of rough ground and then down the shaft into the heading. On account of cold weather the entire pipe line was given about 3 ft. of earth cover. There were about 25 sags in the line with drain cocks at the lowest points. These low spots were exposed, and during freezing weather constant fires were provided to keep them free.

The steel was sharpened with a Sullivan, S-55 drill sharpener, using a coal furnace.

The drilling and trimming crews per shift consisted of 1 foreman, 3 drillers, and 3 dummies or helpers. Mucking crews were made up for each heading as follows:

	East Heading	West Heading
Mucking machine operators	1	1
Muckers	4	4
Electric dinkey operators	1	..
Brakeman	1	..
Dumpmen	1	3
Mule skinners	..	2
Cagemen	..	1
Hoisting-engine operators	..	1

During January 1929, when the heading had advanced about 2 000 ft. at the east portal, observations were made for a few days and the following data obtained:

Actual drilling done during 11½-hour shift, 8 hours and 7 minutes. Drilling at the heading 29 ft. per drill per hour. It took:

48 minutes to load 26 holes in heading and to shoot the cut.
38 minutes to reload the cut, wire and shoot the heading.
18 direct and 17 delays per shot.
3 minutes to load 1-cu. yd. car by mucking machine.
4 minutes to switch 3 cars by 5 men.
1½ minutes to dump 1-cu. yd. car by 2 men.
50 minutes to lay one 15-ft. section of track by 5 men.

The mucking machine was idle 4 minutes between loading successive cars.

The work was shut down 10 per cent. of the time due to breakdown and repairs of the equipment.

A very careful analysis of actual costs of excavation, made in January, 1929, showed the following, not including overhead or profit to the contractor.

ACTUAL COST PER CUBIC YARD EXCAVATION.

Item.	Tunnel.		Shaft.	
	$	%	$	%
Labor	6.43	69.6	6.21	71.6
Explosives	1.09	11.8	0.55	6.3
Power	0.49	5.4	0.37	4.3
Fuel	0.07	0.7	0.19	2.2
Equipment	1.16	12.5	1.35	15.6
Total	9.24	100.0	8.67	100.0

For the labor item the wages per hour were: Foreman $1.10; hoisting engineer $1.00; drillers $0.70; mucking machine operator $0.75; electric dinkey operator $0.60; unskilled laborer $0.50; blacksmith $0.90; helper $0.60; compressor man $0.70; and carpenter $1.00.

CONCRETE.

The mix for the concrete was 1 bbl. cement weighing 376 lb., 9 cu. ft. sand measured loosely, and 16 cu. ft. ballast measured loosely. The cement, in addition to conforming to the specifications of the American Society for Testing Materials, was required to have less than 1½ per cent. SO_3 content. The sand was to be of well graded, hard durable stock all passing, when dry, a screen having ¼-in. diameter holes and not more than 30 per cent. by weight passing a sieve having 50 meshes per linear inch. The ballast was required to be of well graded, hard, durable stock with no particle larger than that which would pass through a 2½-in. diameter ring and with all particles smaller than ¼-in. removed. The mixing time was three minutes per batch. An average of 6½ gallons of water was used per bag of cement.

The tunnel muck was not suitable for concrete aggregates. The contractor installed a Farrell crushing plant with a rotary screen in a gravel borrow pit near the east portal. The borrow pit was a glacial deposit containing many hard cobbles, which when crushed provided the coarse aggregate. There was an abundance of sand, in fact about 50 per cent. of the sand that came through the hoppers had to be wasted. The borrow pit area was stripped to a maximum depth of about 4 ft. and the aggregates, without being washed, were clean enough to be used in the concrete. A 15-ton Northwest crane, with ¾-cu. yd. clamshell bucket, did the excavation, casting the material several times when necessary, and piled the material onto a platform, where about 3 men fed the jaw of the crusher by shoveling by hand. They also picked out mica schist stones found in the material.

The crushing plant worked two 10-hour shifts per day, each crew con-

sisting of 1 foreman, 1 crane operator, and about 8 laborers. The average costs per cubic yard to the contractor were about $1.50 for gravel, and $1.00 for sand.

For pouring the concrete from the east portal, the mixing plant was located near the portal. The concrete was mixed in a 4-bag batch, electric operated Ransome mixer, and discharged directly into 1 cu. yd. end-dump wooden cars. Trains of up to 4 cars were hauled by small Plymouth gas and Mancha electric dinkeys over the narrow-gage track to the place of pouring.

The invert was poured first, then the side walls to the springing line, and then the arch. During the pouring of the invert, the night crew thoroughly cleaned the bottom of the tunnel to solid ledge for a sufficient

FIG. 11.— ARCH FORMS IN PLACE.

distance ahead for the concreting crew during the next day. No backfilling with muck, even in large overbreaks, was allowed. About 6 000 ft. of invert were poured from the east portal concrete plant. The general average progress per shift for the invert was about 110 ft., with approximately 35 cu. yd. of concrete, although as many as 240 ft. of invert per shift were poured.

About 5 000 ft. of side walls and arch were poured from the concrete plant at the east portal. This concrete work was started at two points about 2 200 ft. apart, progressing towards the east portal from both places. Collapsible, steel-lined forms, both for the side walls and the arch were made on the job in 50-ft. sections. The side walls were poured always three or four sections ahead of the arch (Figs. 10 and 11).

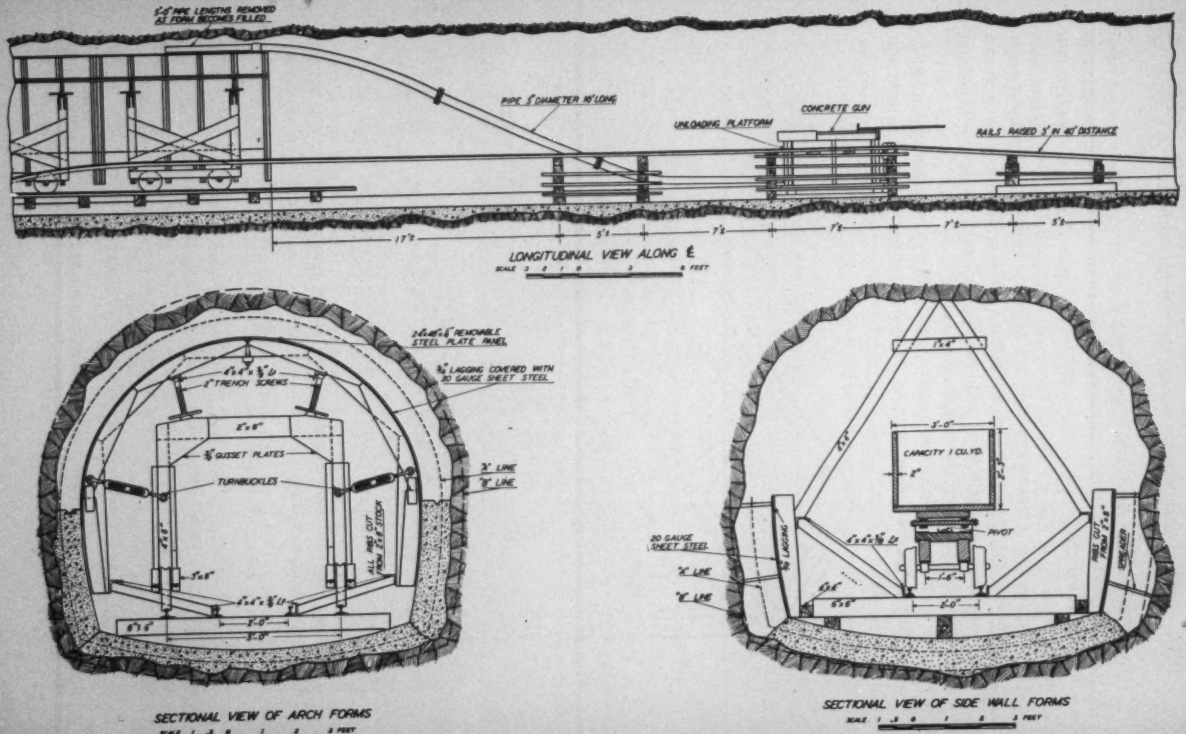

FIG. 10.— CONCRETE LINING FORMS USED IN TUNNEL.

The concrete for the side walls was shoveled from the cars by hand onto chutes leading back of the forms. The arch was shot by pneumatic process with about 70 lb. per sq. in. pressure, using a ⅓-cu. yd. concrete gun (Fig. 12).

In the mix of the concrete for the arch, the contractor, without extra charge, added 0.7 bags of cement per cubic yard of concrete so as to have a more workable concrete than could be obtained with the specified normal mix without increasing the cement-water ratio.

A crew consisting of one foreman and eight laborers was continuously kept busy with the forms, stripping, moving, and re-setting. The concrete

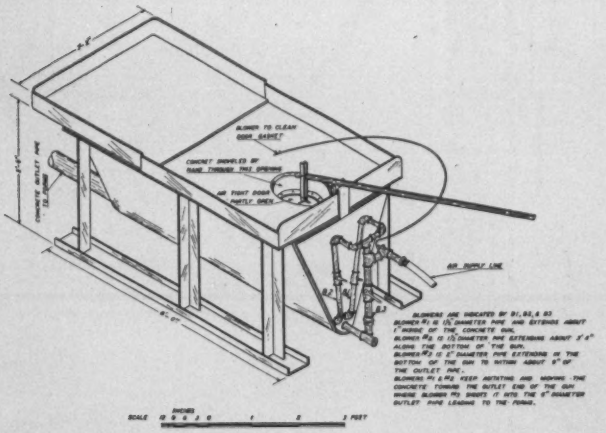
BLOWER TO CLEAN DOOR GASKET
CONCRETE SHOVELED OR RAKED THROUGH THIS OPENING
AIR TIGHT DOOR PARTLY OPEN
AIR SUPPLY LINE
SCALE

FIG. 12.— CONCRETE GUN OF ⅓-CU. YD. CAPACITY.

crew consisted of one foreman, one concrete gun operator, and about six laborers.

Each concreting crew, during two shifts, poured a 50-ft. section of side forms and arch, thus pouring about 100 ft. of lining, exclusive of invert, per day.

It took, on an average:

3½ hours to pour 50 ft. of side walls, 17 cu. yd.
5 hours to strip, clean, oil, and reset 50 ft. of side forms.
7 hours to pour 50 ft. of arch form, 35 cu. yd.
4 hours to strip, clean, oil, and reset 50 ft. of arch forms.
10 hours before stripping the side wall forms.
10 to 12 hours before stripping the arch forms.

A second mixing plant was located at the top of the shaft for concreting the lining of the shaft and the tunnel from the west portal, Station 8+85

to Station 26 + 00. A two-bag batch, electric driven, Ransome mixer was used. The concrete was dumped into a rectangular open chute leading into a 6-in. diameter pipe down the shaft to the bottom. Here the concrete was discharged directly into 1-cu. yd. cars and pushed by men a maximum distance of about 1 500 ft.

It was soon found that the two-bag batch mixer did not have sufficient capacity, and that the 6-in. diameter pipe chute down the shaft was too small, plugging quite often. They were replaced by a similar type four-bag batch mixer and an 8-in. pipe chute.

The main shaft and the air-vent shaft, including the landings, were poured in lifts ranging from 5 to 10 ft., with 66 to 220 cu. yd. respectively. Construction joints, as a rule, coincided with the elevation of the successive rack support frames. The concrete was placed directly from the chutes, a length of chute pipe being taken off as the concrete elevation came up higher in the shaft. Special care was taken to make the construction joints watertight in the partition wall between the main shaft and the air-vent shaft. For this purpose ¼-in. thick, 4-in. high, and 9 ft by 10 ft. welded rectangular steel bands were used.

The contractor was permitted to use sand bags for bulk heads between the forms and the rock at the end of each section of the forms. He was, however, required to put in additional 2-in by 4-in. keys. This method of bulk-heading is very satisfactory, and its cost is negligible compared with the wooden plank bulk heads that were used for the diversion tunnel sections.

It was almost impossible to hold the forms true to line, due perhaps to not enough bracing, and especially to the vibration caused by the concrete trains going through the jumbos of one set of forms.

The job was good except that the construction joints at the springing line always had an offset, sometimes up to ¾-in. This offset had to be chipped high enough, repaired and finished in order to produce the effect of a curved surface. This required the constant time of two finishers. For construction joints 1:2 mortar was used. For patching up and for repairs a mix of one part of sand and one part of cement was used. In some cases, when water seeped through the spot, the patch would not hold, and a mix of water glass with neat cement was used for quick setting and satisfactory finish. The water-glass mix took about one-half minute for setting completely.

Briquettes were made daily in the field, and a complete record was kept. The average strength of all the field briquettes was 262 lb. per sq. in. for 7 days and 357 lb. per sq. in for 28 days; a few cylinders made for compression tests, giving values from 2 690 to 3 530 lb. per sq. in. for 28 days. The average ratio between the tensile and compressive strengths was found to be 9.2.

Tunnel Pantograph.

A new tunnel pantograph (Fig. 13) was designed and made on the job for a ratio of 30 to 1, consisting mainly of the following parts: (1) the base of 1½-in. in diameter and 2 ft.-8-in. long steel pipe supported by a plate and three 1-in. diameter legs, each leg being held by ½-in. diameter pipe for

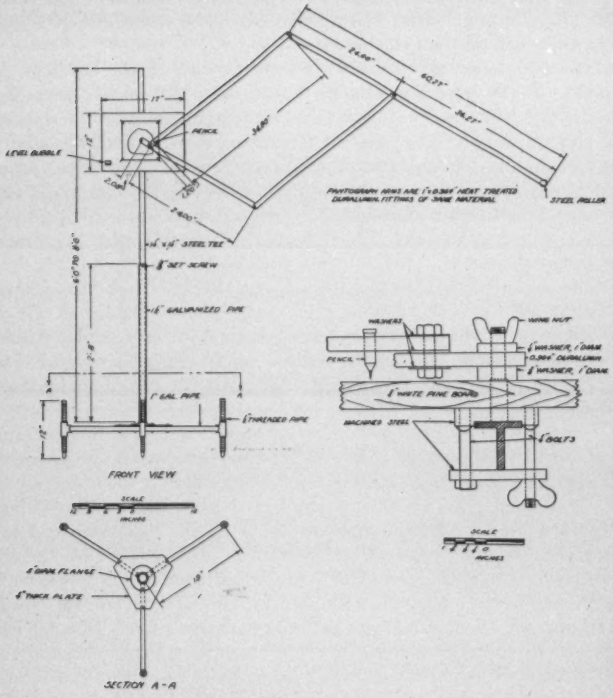

Fig. 13.— Tunnel Pantograph.

leveling the instrument; (2) the stem of 1¼-in by 1¼-in. steel "T" section 6 ft. long, held firmly inside the base at any desired elevation; (3) the board of 1-in. thick white pine, 12 in. by 17 in., with necessary fittings and a level bubble, held on the stem at any height; (4) the pantograph arm and fittings, made of 1 in. by 0.364 in. heat-treated duralumin, supported on the center of the board.

Pantograph sections were taken of the bore every 25 ft., and more often

when necessary, throughout the entire length of the tunnel in order to determine the exact quantity of the excavation and also the shape of the sections. These data were used also in computing the actual concrete placed in lining compared with payment quantities.

The required minimum excavation and thickness of concrete lining, and also payment lines are shown on the typical section (Fig. 4).

COMPARISON OF ACTUAL AND PAID QUANTITIES PER LINEAL FOOT OF TUNNEL.

	Cu. Yd	Per Cent.
Excavation:		
Minimum ("A" line)	3.57	...
Payment ("B" line)	4.50	...
Actual	4.31	...
Payment less actual quantities	0.19	4.4
Concrete:		
Minimum ("A" line)	0.645	...
Payment ("B" line)	1.57	...
Actual	1.38	...
Payment less actual	0.19	12.1

GROUTING.

Before the lining was started a careful survey was made of the stations of running water, large seams in the rock, and also the mud pockets. Drain pipes were placed through the forms to take care of the running water while pouring the concrete. Later on these pipes were grouted. The purpose of grouting was to fill up the space left between the concrete lining of the tunnel and the rock. Grout holes were put in the center of the roof of the tunnel every 20 ft. Every fourth hole was grouted first. After allowing 48 hours for the grout to set, the holes half way between, that is 40 ft. center to center, were grouted. These also were allowed 48 hours to set. Then the remaining holes were grouted.

The first set of holes was grouted starting with about 10 lb. per sq. in. pressure and increasing it up to about 50 lb. per sq. in., or until grout started flowing freely in the neighboring holes. The same procedure was followed with the half-way holes, except that the pressure was increased up to 100 lb. per sq. in. in case no grout appeared in some other holes before then. The final holes were grouted in all cases up to 100 lb. per sq. in. pressure. In the final holes, however, in case the water, if any, did not stop coming through the grout hole with a pressure of 100 lb. per sq. in., the grout pressure was increased up to a maximum of 150 lb. per sq. in.

The amount of grout taken in any hole back of the lining in a mica schist tunnel is no criterion of the void space between the concrete and the rock in a section, say 10 to 15 ft. away. The assumption was that if the tunnel were thoroughly grouted every 20 ft., all the voids would be filled, and that the amount of grout taken in the last holes would indicate whether or not additional intermediate holes were to be put in. In many sections of the tunnel the adjacent holes took considerable grout, and yet intermediate

holes took as much. On the other hand, in some sections of the tunnel the 20-ft. distant holes took no grout, and yet intermediate holes did take some grout. It was concluded that the grout was stopped by uneven breaks or juts of the rock in passing from one hole to another at 20-ft. distances. Undoubtedly, in some cases, grout did travel a great deal more than 20 ft. as evidenced by the grout seeping through the concrete lining at some farther point where the lining was honeycombed. The water, however, seemed to travel farther. As grouting continued in the nearby locations, where apparently the concrete was in first-class condition, the water started seeping through. The real problem was to determine whether all the void space between the concrete and the rock was filled up with grout. This could not be ascertained by the quantity or the lack of grout taken in any hole. This problem was solved by the help of the pantograph sections, from which the necessary yardage of concrete was computed for different sections. By knowing the exact quantity of concrete actually placed within those sections from careful records, it was possible to determine how much space was left in that particular section to be filled up with grout. As the water came through at different places in the lining after the grouting in the crown was completed, additional grout holes were put in those wet spots. There were 103 such holes, about 50 per cent. taking grout, some of which took as many as 77 bags.

The grout pipes were 1½ in. in diameter and 4 in. long with a 2-in. sleeve at the bottom (Fig. 14). One thickness of a 2-in. wide strip of cement bag dipped in grout was wound around the pipe and forced into the hole of the concrete lining. To the sleeve at the bottom of the grout pipe was attached a valve which was connected to the grout hose of the grouting machine. When the grouting was completed for any hole, the valve at the bottom of the sleeve was closed before the grouting hose was disconnected. The grout in the hose was discharged back into the grout chamber through a bypass near the grout machine. Thus no grout was wasted, and no time was lost in cleaning out the grout hose.

Two Worthington pattern, duplex piston, 6 in. by 4 in. by 6 in., air-driven pumps were used as grouting machines (Fig. 14). They were mounted on flat cars and hauled over the narrow-gage track in the tunnel to the places of grouting. On each flat car was fitted a mixing chamber 2 ft. by 4 ft. by 3 ft. The grout batches consisted of 2 bags of cement and about 13 gal. of water, which were mixed by hand for about 3 minutes. The mix was also agitated by going through the pump into the mixing chamber over and over again.

After the grout was set, the grout pipe and the valve were removed and cleaned thoroughly to be used over again. About 4-in. of opening in the concrete lining below the grout were filled up with concrete and pointed.

The contractor had the option, in holes taking more than 1 bag of cement, to use grout mix of 1 part cement and 2 parts of clean fine sand. However, he decided on using entirely neat cement with about 1 to 1 mix. With a neat cement mix, the grout will travel farther and creep in places

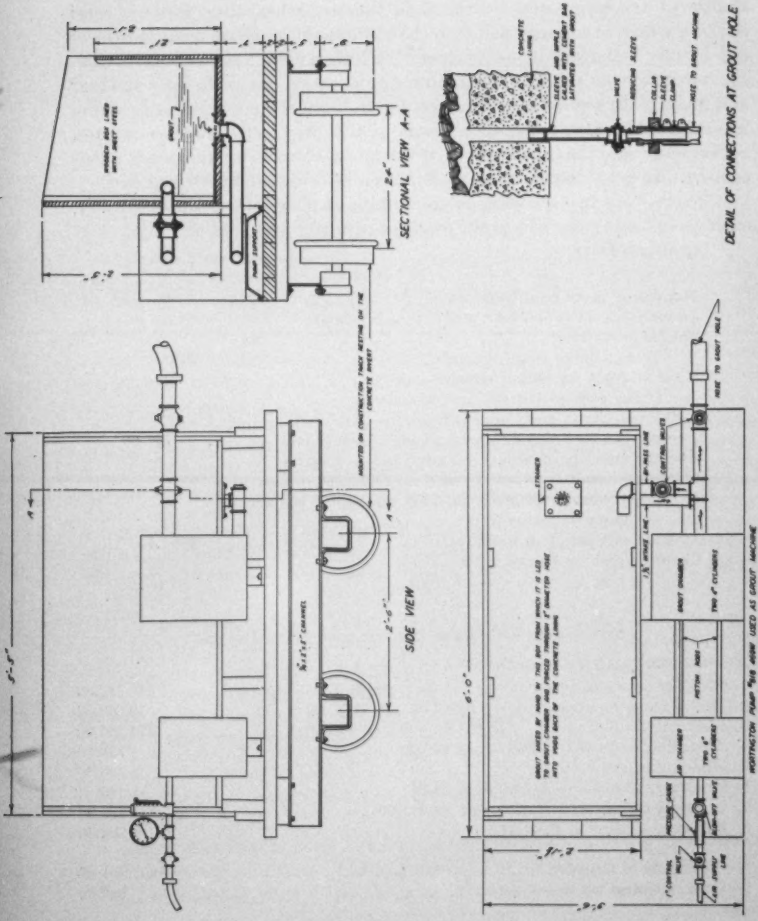

FIG. 14.—CONTRACTOR'S GROUTING EQUIPMENT.

more easily filling the honeycombs and voids in the concrete than the grout and sand mixture. In the case of grouts containing 2 parts sand and 1 part of cement, often the sand, due to heavier weight, will stop during the discharge of the grout and let the fluid travel farther, thus forming sand pockets which eventually will stop the passage of the grout coming in later on. It may look as if the neat cement grouting will be more expensive than the one mixed with sand. Taking into consideration the extra wear and tear and more often repairs of the pumps due to sand, the possibility of putting in additional grout holes on account of the previously mentioned sand pockets and also the time element, it will be cheaper in the end to use a neat cement mix even though a few more barrels of cement may be required.

There were two grouting gangs working two 11½-hour shifts per day, each group consisting of 1 grout-machine operator and 2 laborers.

Grouting Data:

Number of grout holes in crown	336	
Grout pipes set in concrete, due to running water	15	
Special grout holes	103	
Total number of grout holes	——	454
Yield of grout per bbl. of cement — cu. yd. (1 bag cement with 6½ gallons water.)		0.2
Total quantity of grout used (1560 bbl.) — cu. yd.		315.7
Average quantity of grout used per lineal foot of tunnel—cu. ft.		1.3
Average quantity of cement per grout hole — bags		13.7
Maximum quantity of cement used in grout hole — bags		237
Total yardage of grout to total yardage of concrete in lining — per cent		3.6
Average cost per grout hole		$24.64
Grouting cost per foot of tunnel		1.75
Grouting cost per cubic yard of grout		35.40

TOTAL COST OF POWER TUNNEL AND GATE HOUSE.

Tunnel, Including Open Cut at Portals.

1. Earth excavation, open cut — 7 741.9 cu. yd. @ $2.00	$15 483.80
2. Rock excavation, open cut — 8 519.9 cu. yd. @ $5.00	42 599.50
3. Rock excavation, tunnel — 32 488.6 cu. yd. @ $10.00	324 886.00
4. Timbering — 14 662 M F.B.M. @ $50.00	733.10
5. Cleaning tunnel bottom	3 136.00
6. Grouting, all types — 454 holes @ 24.64	11 185.42
7. Concrete lining — 10 621.0 cu. yd. @ $18.00	191 178.00
8. Bonus — 37 days @ $100.00	3 700.00
Total cost of tunnel	$592 901.82
Cost of tunnel per lineal foot	$83.70

Shaft, Including Approach Channel.

1. Earth excavation, open cut — 5 154.9 cu. yd. @ $2.00		$10 309.80
2. Rock excavation, open cut — 178.0 cu. yd. @ $5.00		890.00
3. Rock excavation, shaft — 3 000.4 cu. yd. @ $7.00		21 002.80
4. Concrete — 2 547.06 cu. yd. @ $18.00		45 847.08
5. Reinforcing steel — 44 309.0 lb. @ $0.05		2 215.45
6. Structural steel:		
Cost, purchase — per lb. @ 6.24 ct.	$7 197.85	
Installation — per lb. @ 3.00 ct.	3 464.02	
Total cost — 115 467.2 lb. @ 9.24 ct.		10 661.87
7. Gate and Appurtenances:		
Cost, purchase — per lb. @ 23.36 ct.	$18 875.00	
Freight — per lb. @ 0.54 ct.	442.08	
Installation — per lb. @ 5.00 ct.	4 054.44	
Additional cost of concrete, 86.8 cu. yd. @ $14.00	1 215.20	
Makers supervision	526.76	
Total cost — 80 793 lb. @ 31.1 ct.		25 113.48
8. Hand railing, access shaft, in place — 77 ft. @ $1.52		117.07
9. 12-in. diameter bypass valve, oil tanks, hand pump, accessories, in place		528.53
Total cost of shaft		$116 686.08

Gate House.

1. Concrete — 250.0 cu. yd. @ $32.00		$8 000.00
2. Cornice — 116.0 ft. @ $3.58		415.95
3. Sills — 55.0 ft. @ $3.28		180.37
4. Reinforcing steel — 19 220.7 lb. @ $0.05		961.03
5. Structural steel — 4 894.0 lb. @ $0.089		434.43
6. Steel sashes, area — 610.0 sq. ft. @ $1.091		665.35
7. Glass and glazing — 252 lights @ 47.27 ct.		119.10
8. Conduits, electric		128.67
9. Spiral stairway, 26 ft., height		548.97
10. Chimney, 20 in. by 24 in. inside, 2nd floor up, 22 ft. high		477.44
11. Roof:		
Steel truss	$810.00	
Porete slabs	541.75	
Ludowici-Celedon slab shingle tiles	1 451.00	
Total — 1 214 sq. ft. @ $2.31		2 802.75
12. Electrical Wiring		885.04
13. Approach to Gate House		3 315.61
Total Cost of Gate House		$18 934.71
Total cost of Power Tunnel and Gate House		$728 522.61

The cost of engineering, Division Engineer's office only, has been about 5 per cent. of the total cost. This included all the preliminary investigations, locations and plans prior to construction; the lines, grades, inspection, testing laboratory, etc., during the construction; and final estimates and record plans after the completion of the job.

The excavation at the East Portal was started on September 11, 1928 and the tunnel was holed through on November 12, 1929. Concreting started on October 13, 1929 and was finished on August 25, 1930.

The work was done by the Board of Water Commissioners of the City of Springfield, Mass., with E. E. Lochridge as Chief Engineer and Hazen & Everett as Consulting Engineers, and the writer as Division Engineer in charge of the construction. Under the writer's supervision, A. V. Lynn and C. Kay acted as Resident Engineers during day and night shifts respectively, and L. P. Hayes had charge of the concrete aggregates and laboratory. The Frazier-Davis Construction Company of St. Louis, Missouri, were the Contractors with R. J. Burns as General Superintendent.

During the construction the members of the Board of Water Commissioners were: Edwin A. Blodgett, Chairman, Dr. George L. Fenn, and Mayors Fordis C. Parker and Dwight R. Winter, ex-officio; James J. Shea served as member during 1932 and as chairman during 1933 with John M. Noonan as member. The present members are Henry F. Punderson, Chairman, John M. Noonan and Mayor Henry Martens, ex-officio.

BOSTON METROPOLITAN WATER SUPPLY EXTENSION.

BY KARL R. KENNISON.*

[*Read February 14, 1934.*]
[*Corrected to April 21, 1934.*]

The development of the Ware and Swift rivers for the additional water supply of the Boston Metropolitan District was started in 1926 and the construction is now about half completed. It is expected that the entire new development will be ready for use about 1940 or shortly thereafter. Following is a synopsis of the subject matter contained in this paper:

Part I. General description of the water supply development.
Part II. Program of the new development.
Part III. Description of the principal construction items.
Part IV. Costs and miscellaneous data.
Part V. Origin of the name "Quabbin".
Part VI. Bibliography.

PART I

General Description of the Water Supply Development.

This Boston district is the fourth largest metropolitan center in the United States, and has a high concentration of population, Massachusetts having the second densest population of any state in the Union with forty-seven per cent. of its people living on seven per cent. of its area. The water district includes Boston and nineteen other cities and towns with a population of 1 500 000, all located within ten miles of the State House. There are fifteen other cities and towns with a combined population of 400 000 which are potential members of the water district. The District now supplies to its members, in normal times, about 140 m.g.d., the chief source of supply being the Wachusett Reservoir with a drainage area which was originally 119 sq. miles but which has been reduced to 88 sq. miles by legislative grants from time to time to the City of Worcester. On the basis of Worcester's reduced present needs, the Wachusett area now available to the District is practically 109 sq. miles. The District also uses regularly a considerable area at the head waters of the Sudbury River, about 22 sq. miles of which are tributary to the Sudbury Reservoir, one of the principal steps in the line of aqueducts which bring the Wachusett water to the District.

The District maintains the necessary distribution reservoirs, standpipes and pumping stations to deliver the water to its member municipalities, who are assessed the entire fixed and maintenance charges as a part of their annual state tax. The cost is proportioned to the different municipali-

* Designing Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

ties, two-thirds on the basis of water consumption, and one-third on the basis of valuation. The Water Division of the Metropolitan District Commission maintains the systems of supply and distribution and will take over the new works upon their completion by the Metropolitan District Water Supply Commission.

There have already been presented before this Association papers describing the present supply and the investigations leading up to the authorization of the extension of the sources to the Ware and Swift rivers. Notable among these is a paper presented February 14, 1922, by X. H. Goodnough, an Honorary Member of this Association, for many years Chief Engineer of the State Department of Public Health and at the time Chief Engineer of the Joint Board which investigated the future water supply needs of the District and recommended extending the sources of supply westward from Wachusett Reservoir to the Ware, Swift and Millers rivers. Mr. Goodnough was untiring in his efforts to secure legislation which would continue the established policy of obtaining water from unpolluted sources with large storage possibilities, and he deserves much credit for the outcome. The water-supply development was also described in a paper by Chief Engineer Frank E. Winsor, a Past President of this Association, published in the December, 1929, issue of the Proceedings of the American Society of Civil Engineers.

The New Supply. The extension of the supply, now under construction, was authorized by Chapter 375 of the Acts of 1926 and by Chapter 321 of the Acts of 1927. To carry out the work, the Governor appointed three commissioners as the Metropolitan District Water Supply Commission, and the actual development commenced in October, 1926, with their appointment of Mr. Winsor as Chief Engineer. The project as authorized, although practically that recommended by the Joint Board in 1921, includes the development of the Ware and Swift rivers only; it does not cover any taking of the waters of the Millers River. Briefly, the plan contemplates the construction, in the valley of the Swift River, of Quabbin Reservoir which will be 18 miles long and hold 415 billion gallons of water, or over six times as much as Wachusett Reservoir. It will have an area of 39 sq. miles and a shore line about 151 miles long, including that of about 60 islands. The depth of water will be about 100 ft. at the water-supply intake, and 405 of the total 415 billion gallons will be available for the District. Tributary to this reservoir will be 186 sq. miles of the Swift River and 98 sq. miles of the Ware River drainage areas.

To impound the water, two large earth dams will be required, the main dam across the Swift River about 2½ miles south of the village of Enfield, and a dike about 2½ miles to the east across the valley of Beaver Brook. The dam will be about 2 640 ft. long, 170 ft. high above the river bed and at least 280 ft. above the surface of sound ledge, and will contain about 4 000 000 cu. yd. above the original surface. The dike will be about 2 140 ft. long, 135 ft. high above the brook bed and 264 ft. above the surface of

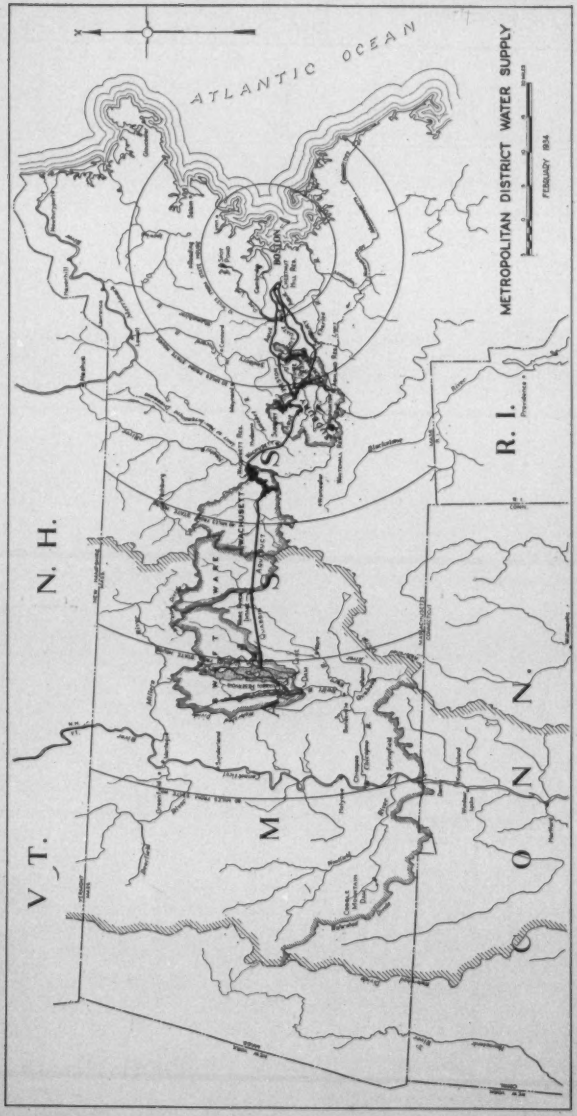

FIG. 1.—METROPOLITAN DISTRICT WATER SUPPLY.

FIG. 2.— QUABBIN RESERVOIR.

sound ledge, and will contain about 2 500 000 cu. yd. above the original surface.

The new Quabbin Reservoir will supply Wachusett Reservoir through a tunnel now nearing completion. Unlike any of the existing aqueducts from the Wachusett supply which consist of alternate adjoining sections of tunnel, cut and cover, siphon pipes and some open channel, the new Quabbin Aqueduct is a single tunnel 24.57 miles long between portal face and outlet shaft, making a continuous tube from the intake structure to the outlet shaft 24.61 miles long. This is comparable in length with a similar tunnel 25.1 miles long which the City of San Francisco is now completing through the Coast Range as a part of its Hetch Hetchy Aqueduct. These

FIG. 3. — WARE RIVER INTAKE WORKS AT TOP OF SHAFT 8, A 260-FT. DROP INTO QUABBIN AQUEDUCT TUNNEL. NOVEMBER 27, 1931.

tunnels are the longest in the world and twice as long as the longest railroad tunnel under the Alps. The only other tunnels comparable in length are two water-supply distribution tunnels of the City of New York extending southerly from its Hill View Reservoir in Yonkers, City Tunnel No. 1, 18 miles long and under Manhattan Island itself, City Tunnel No. 2, 20 miles long now under construction through the Bronx, Queens and Brooklyn to a shaft near Governors Island, and the Shandaken Tunnel 18.1 miles long, also a part of New York's water-supply system.

The Quabbin Aqueduct tunnel will tap the bottom of the new reservoir at a point in the town of Greenwich and conduct the waters of Quabbin Reservoir by gravity to the upper end of Wachusett Reservoir in West Boylston. This tunnel will have no gates at the Quabbin Reservoir intake, but its discharge into Wachusett Reservoir will be controlled entirely by

gates at the east or Wachusett Reservoir end. Hence the tunnel will be in effect a part of Quabbin Reservoir itself; the water level will rise in all the shafts and be governed by the level in Quabbin Reservoir, and any water discharged into the tunnel will be free to flow westward back into Quabbin Reservoir.

At about the halfway point the tunnel passes beneath the Ware River at a depth about 260 ft. below the spillway of the Ware River intake works below the village of Coldbrook, so that the Ware River is made tributary to Quabbin Reservoir by the drop into the tunnel at that point. Furthermore, the Ware River intake is high enough so that floods twice as great as any observed in the period of record during the last 20 years can be dropped into the tunnel, and there is sufficient head to force them the entire 11.37 miles westward into Quabbin Reservoir and 13.24 miles eastward into Wachusett Reservoir. In other words, on account of this high level of the Ware River intake works directly above the tunnel it was not necessary to construct any storage reservoir on the Ware River to prevent spillway waste, the Quabbin Reservoir serving to store the run-off from the Ware River drainage area just as effectively as that from its own Swift River drainage area; nor, for the same reason, was it necessary to build the tunnel any larger to deliver the high rates of flood flows from the Ware intake than would in any event be required for the through delivery from Quabbin Reservoir to Wachusett Reservoir.

Towns Flooded by the Reservoir. Several small villages are located in the area to be flooded by Quabbin Reservoir. These will be wiped out in the course of the construction. The area involved, however, is so large that unprecedented measures will be put into effect under the provisions of the legislation which authorizes a realignment of both town and county lines. The towns of Enfield, Greenwich and Prescott will not only be largely flooded but will be wiped out of corporate existence, and portions of each will be annexed to the adjoining towns. After the realignment, the lines will in general follow the water courses so that regardless of the extent to which the reservoir is filled at any time, the dry area of each town will be undivided by the waters of the reservoir. The legislation providing for such drastic changes contains many interesting provisions to protect the rights of individuals as citizens of these towns and counties. It will be several years before this realignment will be made effective; however, the Commission took over the entire administration of the town of Prescott July 25, 1928. The population on the entire watershed of Quabbin Reservoir in 1925, before the work on the new development commenced, was only 21 per sq. mile. Since 1850 there has been a general decrease in the population on the area tributary to the site of Quabbin Reservoir.

The character of the land to be flooded is approximately 27 per cent. arable, 4 per cent. pasture, 5 per cent. swamp and water surface, 21 per cent. pine, 15 per cent. other woodland and 28 per cent. sprout land and mixed growth.

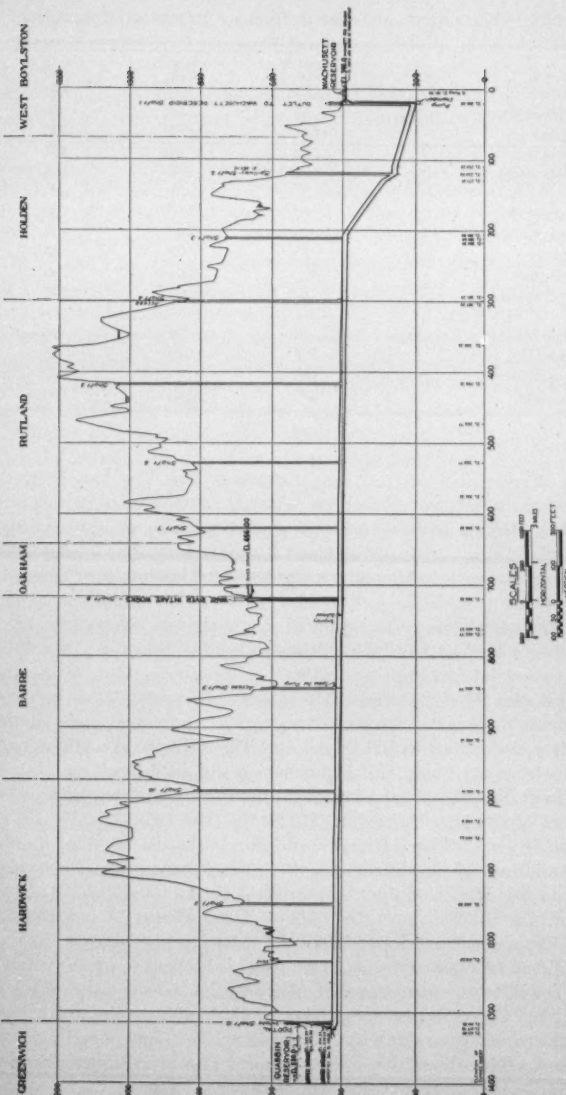

FIG. 4.—PROFILE OF QUABBIN AQUEDUCT.

TABLE 1.— POPULATION OF TOWNS IN QUABBIN RESERVOIR WATERSHED.

Town.	Estimated Population of Town Within the Watershed. Census of 1850.	Census of 1925.	Census of 1930.
Barre	150	150	170
Belchertown	45	50	60
Dana	842	657	505
Enfield	1 036	749	497
Greenwich	838	450	208
Hardwick	70	110	90
New Salem	1 080	444	400
Orange	10	25	25
Pelham	491	260	200
Petersham	1 222	547	550
Phillipston	56	25	25
Prescott	737	230	48
Shutesbury	456	104	100
Wendell	276	132	100
Total	7 309	3 933	3 008

The Watershed Areas and the Connecticut Case. The two rivers, the Ware and the Swift, from which the water is taken, are tributaries of the Connecticut River. Together with the Quaboag River, whose area adjoins them on the south, they unite at Three Rivers to form the Chicopee River which is highly developed industrially and which flows into the Connecticut River just above Springfield. There is no present industrial development on the Ware River above the point of diversion, but below there are, in addition to a mill at Coldbrook whose site was taken for the intake works 9 industrial developments on the 31 miles of river downstream which have developed 187 of the total 330 ft. of drop in the river. These industries will continue to have the benefit of the intervening drainage area tributary to the river, amounting to 121 sq. miles at Three Rivers, in addition to the low flows from the 98 sq. miles from which the flood flows are diverted. On the Swift River there is no important industrial development above the main dam of Quabbin Reservoir. Below the dam only 16 sq. miles contribute their yield at Three Rivers in addition to the 186 sq. miles diverted. Only the industrial development of the Otis Company at Bondsville is located on this stretch of the river, using 43 of the total 75 ft. of drop in the river. On the Chicopee River below Three Rivers 14 industries are affected, which have developed 232 of the total 260 ft. of drop in the river. An additional tributary area of 211 sq. miles is brought in by the Quaboag River. The total tributary area is increased by 88 sq. miles along the course of the Chicopee River, making a total of 720 sq. miles at its mouth before diversion. These facts as to the geographical location and industrial development have been the primary considerations in determining the nature and extent of the development of the new sources.

The only point on that portion of the Connecticut River affected by the diversion at which the natural head is developed for industrial purposes is at Windsor Locks. Here, although there is a potential development of about 50 000 h.p. at 30-ft. head, the present development by the canal on the west bank is unimportant. However, the interest of the War Department in maintaining navigation on the lower Connecticut River made it necessary to obtain from that department permits for the Ware and the Swift diversions. On January 16, 1928, the State of Connecticut filed a bill of complaint with the United States Supreme Court to enjoin the Commonwealth of Massachusetts from making the proposed diversion of waters of the Ware and Swift rivers. The Court's decision on February 24, 1931, denied Connecticut's contention and upheld the right of Massachusetts to make the diversion subject to regulations of the War Department. A paper describing this case was presented to this Association by Chief Engineer Winsor on October 1, 1931, at which time a paper describing a similar case involving an interstate diversion from the Delaware River for the water supply of the City of New York was presented by Thaddeus Merriman.

Yield of the New Supply. By Massachusetts legislation, the District is authorized to take from the Ware River at Coldbrook only the flood flows in excess of 85 m.g.d.; that is, on any day when the natural flow from these 98 sq. miles does not exceed this rate, which is about 1.34 c.f.s. per sq. mile, no diversion can be made. Furthermore, this diversion cannot be made in the dry months between June 1 and December 1 except with the permission of the State Department of Public Health. By Massachusetts legislation, the entire flow from the Swift River area may be diverted except that a sufficient quantity must be released from the reservoir to insure the maintenance of a flow of 20 m.g.d. into the mill pond of the Otis Company at Bondsville located about 5 miles downstream from the main dam. This means that regardless of what the natural flow of the river may be, there must be discharged each day from the reservoir at the main dam a flow of 20 m.g.d. less the yield of the small intermediate drainage area of about 8 sq. miles above Bondsville and below the main dam.

In addition to these restrictions placed upon diversion by Massachusetts legislation, the War Department, in the interest of navigation on the Connecticut River, does not permit any diversion to be made from the Ware River between June 15 and October 15 and requires under certain conditions of drought a release of additional water from Quabbin Reservoir. The necessity of such release is to be determined by the flow in the Connecticut River as measured by the U. S. Geological Survey at Montague, Mass. On any day when the flow of the Connecticut River at this point is less than 4 900 c.f.s., which is about 0.616 c.f.s. per sq. mile, the District must discharge from the Quabbin Reservoir at the main dam 70 c.f.s.; furthermore on any day when the flow is still lower or less than 4 650 c.f.s., which is about 0.585 c.f.s. per sq. mile, the District must discharge 110 c.f.s. These releases are not in addition to those required to guarantee the flow at

Bondsville, but are the total required releases. The restrictions imposed by the War Department reduce the supply available from the Ware and Swift rivers to about 93 per cent. of the maximum that would have been available under Massachusetts legislation, and require approximately 7 billion gallons of storage capacity in which the quantity of water required to be released during periods of drought can be stored over a long period of years.

It is estimated that the additional safe yield, during a prolonged drought, available to the District from these new sources is 210 m.g.d., which means the safe yield of the present sources of good quality in the Wachusett and northern Sudbury areas will be multiplied by about 2¾. It is interesting to note that of this total additional yield, 40 m.g.d. come from the Ware River and 180 from the Swift River, from which must be subtracted a loss of about 25 m.g.d. in evaporation from the large area of water surface in the new reservoir; and about 15 m.g.d. come from a saving of waste over the Wachusett spillway on account of the greater opportunity for economical operation of the two reservoirs in series. This additional yield is being made available in progressive steps, the first being from the Ware which was diverted in 1931 directly into Wachusett Reservoir through the easterly section of the tunnel. However, on account of limited storage available in Wachusett Reservoir, the additional yield obtainable in this way during a critical dry period does not exceed 30 m.g.d., and no large additional yield can be obtained until the tunnel is completed to Quabbin Reservoir.

Emergency Requirements. On account of the shortage in the supply which was threatening at the time the Water Supply Commission was created, this commission was authorized by Chapter 111 of the Acts of 1927 to further develop certain sources of the District in the southern Sudbury area by the construction of pipe lines from Whitehall, Hopkinton and Ashland reservoirs to the reservoirs and aqueducts of the northern Sudbury system. The development of this emergency supply was practically completed late in 1927. It is not an integral part of the new Ware-Swift supply and needs no further comment. Some of the interesting features in connection with this work, including the design of a pipe line located about 26 ft. above the hydraulic gradient, were described in a paper by the author, published in the March, 1933, issue of this JOURNAL.

The level of the Wachusett Reservoir had dropped to 32.3 ft. below its spillway level in January, 1927, and continued approximately 12 ft. lower than in 1926 until in the fall of 1927 heavy rains, unusually heavy for that time of year, not only caused the Vermont flood but insured the filling of Wachusett Reservoir during the next spring. As a result, these pipe lines from the emergency southern Sudbury source were not operated except for test purposes until January, 1930. Thereafter they were used effectively throughout the drought of 1930 and until the completion of construction of the easterly section of the Quabbin Aqueduct tunnel enabled the District to begin the diversion of flood flows from the Ware

River in March, 1931. This was just after the Wachusett Reservoir had dropped to an all-time low of 45 ft. below its spillway level, in February, 1931, and there were only 21 billion gallons left in storage, of which only about one-half was available for draft through the Wachusett Aqueduct. The additional yield obtainable from the Ware River by the completion of this first stage of the development was most timely.

Worcester's Needs. The needs of the city of Worcester for an additional supply of water have been closely tied to those of the Metropolitan District throughout the entire history of the development of these new sources. The Joint Board in 1921 proposed that Worcester be allowed to pump out one of the shafts of the tunnel into her Pine Hill Reservoir drainage area. The report of the special investigating commission in 1926 suggested an even closer association between Worcester and the District in the development of the upper Ware River drainage area. The legislation which created the present water-supply commission authorized the payment of one million dollars by Worcester toward the cost of the District's new supply, in return for which Worcester would be allowed to develop the Quinapoxet Pond area in the upper Wachusett watershed, totaling about 21 sq. miles. Worcester has built a pumping station and a force main from this area into her Kendall Reservoir, but before this was completed she found it necessary, in 1930, on account of the very low level of Wachusett Reservoir, to install emergency pumps and to dam temporarily a small inlet of Wachusett Reservoir in order to increase her facilities for supplementing her supply by pumping from the Wachusett Reservoir.

For the joint benefit of Worcester and the District, the Commission was also authorized by Chapter 262 of the Acts of 1932 to construct a trunk sewer through the towns of Rutland and Holden to a connection with the sewerage system of the City of Worcester for the purpose of maintaining and protecting the purity of the water supplied and to be supplied from the Wachusett Reservoir, the Quinapoxet River and the Ware River.

Legislative Authorization for the New Supply. The various duties of the Metropolitan District Water Supply Commission as outlined in the preceding are specified in certain Acts of the State Legislature. The total amount authorized by the Legislature to be spent for the extension of the water-supply sources of the District to the Ware and Swift rivers, including the southern Sudbury emergency supply, is $65 900 000. Worcester's payment of $1 000 000 was contributed toward this total sum leaving $64 900 000 to be raised by bond issues of the Commonwealth. Bond issues to date aggregate $28 500 000. The enabling acts and the amounts authorized may be summarized on page 158:

The comparative cheapness of this unfailing supply of pure water which is so essential to the very life of the District can be appreciated if we note that the total cost per capita, to the population of the cities and towns in the District, of both the old and the new supply is about 9/10

of a cent a day, and is not expected at any time to exceed a cent a day. This is the wholesale cost which the cities and towns in the District are assessed as a part of the State tax and covers all bond requirements of the old supply, largely financed by 40-year sinking-fund bonds, all bond requirements of the new supply financed by 30-year serial bonds, and all maintenance costs to the point of delivery to the various municipalities. In addition, the individual consumer pays to his city or town its rates for maintenance and distribution. The old supply and the entire distribution system cost nearly $50 000 000 and is 78 per cent. paid for. For the new supply nearly $30 000 000 have been expended to date, and this supply is about 15 per cent. paid for.

TABLE 2.— LEGISLATIVE AUTHORIZATION FOR NEW SUPPLY.

No.	Year.	Chapter.	Principal Features.	Amount Authorized.
1.	1926	375	Water Supply Commission, Ware Supply, Purchase of Real Estate for Swift Supply.	$15 000 000
Amended by:				
1-A.	1931	340	Quinapoxet Grant to Worcester.	..
2.	1927	111	Southern Sudbury Emergency Supply.	plus 900 000
3.	1927	321	Swift Supply, Modification of Town and County Lines.	plus 50 000 000
Amended by:				
3-A.	1928	340	Management of Town of Prescott.	..
3-B.	1934	214	Board of Referees.	..
4.	1931	66 (Resolves)	Investigation of Watersheds by Joint Board.	..
5.	1932	262	Rutland-Holden Sewer.	..
			Total Authorized	$65 900 000

During the District's first 25 years, before construction of the new supply was begun, the assessments on the member cities and towns averaged about 6/10 of a cent per capita per day. The cost of the new supply has increased this gradually to the present cost of about 9/10 of a cent and will maintain it at about this figure during the next ten years, after which it will gradually decrease in another ten years to the cost which prevailed before the present additions were undertaken. Table 3 shows the costs included in the annual District assessment for 1934 compared with those for 1926. The assessments on account of the new supply will be offset to a great extent by the facts that after next year there will be a material reduction of the total bond requirements for the old supply and that by the year 1950 they will have been reduced from $1 754 377 to $145 073.

TABLE 3.— WATER DISTRICT ANNUAL ASSESSMENTS.

	April 1, 1926.	April 1, 1934.
Bond Requirements for the Old Supply		
Payments into Sinking Fund	$174 691	$135 467
Interest on Sinking Fund Bonds	1 393 435	1 393 435
Payments on Serial Bonds	115 000	114 000
Interest on Serial Bonds	147 866	111 475
Total Bond Requirements	$1 830 992	$1 754 377
Miscellaneous Interest Adjustment	12 561	35 000
Additions to the Distribution System	300 000	300 000
Maintenance	769 799	857 624*
Total for the Old Supply	$2 913 352	$2 947 001*
Bond Requirements for the New Supply		
Payments on Serial Bonds		$884 000
Interest on Serial Bonds		898 908
		$1 782 908
Premium on Sale of Bonds		24 330
Total Bond Requirements		$1 758 578
Miscellaneous Interest Adjustment		4 812
Total for the New Supply		$1 763 390

* On the basis of maintenance charges for 1933 which are anticipated to be equalled in 1934.

Personnel. The Commissioner of the Metropolitan District Commission is, *ex officio*, the Chairman of the Metropolitan District Water Supply Commission. The two Associate Commissioners are appointed by the Governor with the advice and consent of his Council to serve for terms of five years. The personnel of the Commission and the engineering departmental heads are as follows:

Commission.

Chairman: Davis B. Keniston, May 28, 1926, to date.

Associate Commissioners: Charles M. Davenport, July 28, 1926, to date.
Joseph H. Soliday, July 28, 1926, to July 29, 1931.
Thomas D. Lavelle, July 29, 1931, to date.

Secretary: R. Nelson Molt, March 3, 1927, to date.

Headquarters Office.

24 School Street, Boston, Mass., October 29, 1926 to December 31, 1930.
20 Somerset Street, Boston, Mass., January 1, 1931, to date.

Chief Engineer: Frank E. Winsor, October 1, 1926, to date.

Designing Engineer and Principal Assistant to the Chief Engineer: Karl R. Kennison, October 16, 1926, to date.

Mechanical Engineer: Walton H. Sears, November 29, 1926, to date.

Consulting Engineers: X. Henry Goodnough, Inc.
J. Waldo Smith (died October 14, 1933).
Chas. T. Main, Inc.

Consulting Geologist: Dr. Charles P. Berkey.

Wachusett-Coldbrook Tunnel Division, Holden, Mass.

Division Engineer: Harold W. Horne, November 1, 1926, to May 23, 1928 (died).
William W. Peabody, July 9, 1928, to date.

Coldbrook-Swift Tunnel Division, Hardwick, Mass.

Division Engineer: Richard R. Bradbury, September 15, 1927, to date.

Quabbin Reservoir Division, Enfield, Mass.

Division Engineer: N. LeRoy Hammond, November 14, 1926, to date.

Southern Sudbury Emergency Supply, South Framingham, Mass.

Assistant Engineer: Walter Knowles, December 20, 1926, to February 29, 1928.

Laboratory

Before Removal to Quabbin Reservoir Division, 175 State Street, Springfield, Mass.
Chemist: Walter H. Junkins, July 23, 1928, to May 1, 1930.

The engineering force, during the last seven years, averaged 125 persons, the maximum, at present, being about 165.

PART II.

Program of the New Development.

It was evident from the start that the item of construction requiring the greatest length of time for completion and subject to the greatest hazards and possible delays was the tunnel. Accordingly, as soon as the engineering force was organized in October, 1926, work was started on the collection of data for preparation of the contract for the east section of the tunnel, the portion extending from Wachusett Reservoir to the Ware River at Coldbrook. The first contract let was for borings with the purpose of locating sound ledge near the surface at suitable sites for shafts from which to carry on the tunnel excavation simultaneously at a number of headings. It was evident that one of these shafts would have to be put down from the high ground in the town of Rutland and that it would have to be 656 ft. deep, considerably deeper than any of the other shafts. Hence a separate contract was prepared as rapidly as possible for sinking this shaft only; this was executed March 30, 1927. It was decided to work the 13.24 miles of tunnel between Wachusett Reservoir and the Ware River at Coldbrook from 8 shafts, No. 1 being at the Wachusett outlet and No. 8 at the Ware River intake, the average distance between shafts being 1.89 miles and the maximum 2.20 miles. To further expedite the work, other shaft contracts were let in advance of the main tunnel contract, one contract for shafts 2, 3 and 4 and another for shafts 6 and 7 being executed on June 1 and 2, 1927, respectively. Shafts 1 and 8, being at the outlet and inlet works respectively, required special designs, and it was not feasible to include them in the other preliminary shaft contracts. An early contract was also executed April 27, 1927, for the construction of a 22 000-volt

electric transmission line along the entire length of the tunnel right-of-way so that power for construction purposes would be immediately available to all contractors. In fact the construction was completed and the line energized in 54 days after the contract was let, so that even the first contractor on the deep Shaft 5 had the benefit of its use.

In the meantime, work was being pushed on the southern Sudbury emergency supply. The entire plan of this development had to be first prepared and submitted to the Legislature; House Document 221, 1927. The Legislature then approved this plan in Chapter 111 as previously noted, and seven contracts were prepared and let and the emergency supply completed ready for use within the year. In the meantime also additional contracts for borings at the sites of the proposed dam and dike for Quabbin Reservoir and for aerial surveys of the reservoir site and along the Ware River were let. Five such contracts for surveys have been executed, one of which, for borings, is still in progress.

The Tunnel Contracts. The first major contract for the tunnel was finally prepared and bids were opened November 18, 1927. It included the sinking of shafts 1 and 8 and the excavation of a heading extending west of Shaft 8 in anticipation of the future extension of the tunnel to the Swift River. This heading was in fact driven 0.95 mile so that the actual length of the Wachusett-Coldbrook Tunnel section, so-called, of Quabbin Aqeduct was 14.19 miles. The bidders were allowed to bid on the easterly half of the work, 6.39 miles, the westerly half, 7.80 miles, or both. Two contracts were awarded. However, both contractors assigned their contracts to one company. Incidentally, a similar option was allowed the bidders on the Coldbrook-Swift section, the subsequent extension of the tunnel, but only one contract was executed. The contract for the Wachusett-Coldbrook section was not executed until April 30, 1928, on account of delays which seemed advisable because of the pending suit brought by the State of Connecticut in the United States Supreme Court. This delay, however, was offset by the fact that the shaft contractors, after turning the headings the required 50 ft. in both directions from the shaft bottoms, were allowed to continue excavating the tunnel headings. By the time the main tunnel contract was let, 1.56 miles of tunnel had been excavated from the 12 headings under the shaft contracts. The tunnel contractors immediately took over the work and continued it with little or no interruption and the excavation was completed ready for lining June 15, 1930.

As the construction of the Wachusett-Coldbrook section of the tunnel neared completion, four contracts were prepared and let for the construction of the Ware River intake works and four for the Wachusett outlet works. These included the construction of the low diversion dam across the Ware River, the construction and equipping of the intake building and the outlet building and the grading of the grounds at these two points. A contract was also let for filling swampy areas upstream from the intake and also one for the construction of a head house at Shaft 4. The construc-

tion was sufficiently completed in January, 1930, to allow the first diversion of the next available flood flows to be made from the Ware River to Wachusett Reservoir, although much of the operating equipment at the intake works was at the time housed in temporary wooden enclosures. The buildings themselves were completed and the grading of the grounds finished late in 1931.

This completed the construction program for the Ware supply except that a contract was executed July 11, 1933, for about 9½ miles of sewer now under construction from the vicinity of Rutland Center through Holden to connect with the Worcester sewerage system, under an agreement with that city. Here also bids on either or both of two portions were invited, but only one contract was let. Some of the interesting features in connection with this work, including the design of a new type of open nozzle, for the measurement of sewage flow, were described in a paper by the author published in the January, 1934, issue of the Journal of the Boston Society of Civil Engineers. These open nozzles were recently tested and rated in the hydraulic laboratory of the Massachusetts Institute of Technology.

The first step in the construction program for the Swift supply was the extension of the tunnel westward to the Swift River to tap Quabbin Reservoir by the construction of the Coldbrook-Swift Tunnel section, so-called, of Quabbin Aqueduct. Since the requirements for speed were not so urgent in the case of this extension west of the Ware intake, the distance between shafts was increased, only four more shafts being required in the 10.38 miles remaining to be excavated. The maximum distance between shafts is 3.17 miles.

The last shaft, No. 12, is located practically at the extreme westerly end only about 320 ft. east of the rock face portal, and at a point where the original surface is 30 ft. below the future flow line of the reservoir. This shaft is designed as a high-level intake to control the draft from the upper 45 ft. of the reservoir also to permit drawing bottom water. The contract includes the construction of about 200 ft. of cut and cover section extending from the rock face portal of the tunnel to a low-level intake in the of the valley. This contract was executed April 23, 1931. The excavation was completed October 16, 1933, and the concrete lining is now being placed. The contract calls for all the shaft details to be completed in April, 1935, and for the tunnel bore itself to be ready for water November 23, 1934.

Preparing Foundations for the Main Dam and Dike. During all this time, studies were being made to determine the best location and method of construction for the main dam and dike of Quabbin Reservoir. The location chosen for the main dam is just north of the Ware town line and not materially different from that suggested by the Joint Board in 1921. The location for the dike is the same as that originally suggested, slightly south of the divide, between the drainage area of Swift River and Beaver Brook, a tributary of the Ware River. Incidentally the small drainage area of

about 1.4 sq. miles between the divide and the dike is included in the previously stated 186 sq. miles of Swift River drainage area.

At both the dam and dike sites, the sound bed rock is covered to great depths with porous glacial material, and a tight cut-off from the original surface to sound ledge will be required. It appeared that such cut-offs could be constructed most economically by sinking a row of reinforced concrete caissons across the valley and sealing them end to end by a method similar to that recently used, in an even more difficult situation, by the Central Maine Power Company in constructing the Wyman Dam at Bingham, Maine. To demonstrate the feasibility of this method, a contract was let September 16, 1931, for the sinking of two exploratory caissons, one at each site. The success of these exploratory caissons and further experiments in lowering the ground-water level at the dike by pumping, paved the way for a contract which was executed December 28, 1932, for the construction of the complete cut-off at the dike. Under this contract a row of caissons has been sunk across the valley of Beaver Brook, and two additional exploratory caissons are being sunk at the main dam for experimental lowering of the ground-water level.

Incidentally, the ability to complete the entire project in successive steps has made possible a particularly thorough study of various methods of constructing the embankments of the main dam and dike of Quabbin Reservoir. These studies and experiments will result, it is believed, in large savings in construction cost. A very large sum was certainly saved on account of the elimination of much of the contractors' risk and uncertainty in bidding on the caisson core wall for the dike, by the preliminary experiments and demonstration of methods and results to be expected.

In the meantime, in anticipation of the construction of a similar cut-off at the dam, the Swift River was diverted around the site of future construction operations through a tunnel in the hill east of the site. A contract for this tunnel was executed August 1, 1931. Materials excavated from the tunnel and the portal cuts were placed where they will later serve not only as toes of the future dam embankment but as a part of the upstream and downstream cofferdams required when the core wall and embankment are built. The construction was completed and the river diverted through the tunnel June 7, 1933.

In the meantime, studies have been in progress to determine the best method of constructing the dam and dike embankments, and field tests are being made by sluicing materials obtained from suitable borrow areas. A contract was executed February 7, 1934, for the exploration of such borrow areas by steam-shovel cuts.

Relocation of Highways, Transmission Lines and Cemeteries. In the meantime, progress has been made in the construction of highways to replace those the use of which will be discontinued by Quabbin Reservoir. Four contracts for this work have been executed, the first on July 20, 1931. New highways constructed to date have a 24-ft. width of bituminous

macadam surface and 3-ft. gravel shoulders. They are constructed with the coöperation of the State Department of Public Works, which contributes the cost of the surfacing and other incidental items, and upon completion become state highways. To date 5.2 miles of such state highway have been completed south of the reservoir on the route from Ware to Belchertown and 20.8 miles are now under construction west of the reservoir, extending from Belchertown through Pelham, Shutesbury and New Salem.

A contract was also executed June 20, 1933, for the relocation of the 66 000-volt electric transmission line of the New England Power Company, which crosses the reservoir site at three points between Leverett and Ware, requiring 8.5 miles of new line around the southerly end of the reservoir in substitution for 6.6 miles. This contract was only recently completed.

The construction of the reservoir will also drown out 15.8 miles of the present location of the single-track Athol Branch of the Boston & Albany Railroad north of West Ware, including 6 railroad stations, 4 steel bridges and 71 culverts over water courses. There are also 18 public and 26 private grade crossings which will be eliminated.

One important problem is that of removing existing cemeteries in the area to be affected by Quabbin Reservoir. A contract was executed October 29, 1931, for the construction of 4 400 feet of driveways to make a new cemetery, known as Quabbin Park Cemetery, in the Town of Ware about one mile south of the dike site. The development covers 18 acres and includes 1 602 cemetery lots, averaging about 320 sq. ft. or space for about 8 graves in each. A winter receiving-vault was also built to hold 8 caskets. This new cemetery was ready for reinterments August 25, 1932, after which no new burials were allowed to be made in the cemeteries to be moved, of which there are 16 containing 5 365 bodies. The work of removal commenced on December 16, 1927. Generally speaking, bodies are reinterred in whatever cemetery the interested parties request. Several have been moved to cemeteries in other states. It is expected that the great majority will be moved to Quabbin Park Cemetery where lots are assigned in exchange for those in the old cemeteries. To date, there have been removed 1 419 bodies and since August 25, 1932, there have been 38 original interments in Quabbin Park Cemetery. Cemetery removals are being made by a force employed directly by the Commission.

Land Takings. The Commission purchased or took by eminent domain land for all its construction requirements, but no general taking has yet been made in the reservoir area. However, the Commission commenced as early as December 7, 1926, to purchase lands offered for sale in this area. Purchases have been generally confined to the area within one-half mile of the proposed flow line, the entire Prescott "peninsula" between the West and Middle branches of the Swift River, and other areas near the tributary streams. It is estimated that about 100 square miles will be ultimately acquired for Quabbin Reservoir.

Of this total, purchases have been made as follows:

Year Ending Nov. 30.	Land Purchased in Quabbin Reservoir Area — Acres.	Under Option at End of Year — Acres.
1927	10 590	4 560
1928	12 350	3 260
1929	10 880	4 020
1930	5 710	2 340
1931	5 450	1 130
1932	3 695	1 000
1933	5 632	1 731
1934 (5 months to April 21)	2 907	471
Total	57 214	

The Commission has also purchased to date 5 277 acres above the Ware River intake for watershed protection and has 314 acres more under option.

The requirements of land for construction purposes have been as follows: For the pipe lines in the southern Sudbury area the taking was generally an easement 30 to 40 ft. wide across private land. An easement was also taken in public highways. For the tunnel, the taking was generally a strip 50 ft. wide taken in fee. In some cases a 50-ft. easement was taken and included the right to construct and maintain not only the tunnel but also an electric transmission line and a road for access on the surface. In case the tunnel line was located where the transmission line could not follow it on the surface, an additional easement was taken for a detour for the transmission line. At each shaft an area was also taken in fee for the disposal of the rock spoil excavated from the tunnel and for the construction of the contractor's camps. The area taken for these purposes was as follows:

At Shaft.	Area Purchased for Construction Purposes — Acres.
No. 1	On Wachusett Reservoir lands.
No. 2	On Wachusett Reservoir lands.
No. 3	26
No. 4	18
No. 5	22
No. 6	22
No. 7	20
No. 8	On Ware Intake lands.
No. 9	20
No. 10	41
No. 11	28
No. 12	On Quabbin Reservoir lands.
Average	25

For the highway relocations, the taking was uniformly a strip 80 ft. wide taken in fee. Slope easements were taken outside this limit where

necessary. For the transmission line relocation, the easement taken was uniformly 125 ft. wide. For the Rutland-Holden Sewer a permanent easement 15 ft. wide was generally taken, and in addition a temporary easement for construction purposes which increased the total width generally to 40 ft. At the embankment fills, deep cuts and other similar locations, the widths of both the permanent and temporary easements were increased as required. Table 4 gives a summary of lands taken by eminent domain for construction purposes to date.

TABLE 4.— SUMMARY OF LAND TAKINGS.

	Acres taken in Fee.	Acres taken in Easement.
Southern Sudbury Emergency Supply	258.8	29.0
Rutland-Holden Sewer	..	22.9
Quabbin Aqueduct	1115.0	24.0
In Quabbin Reservoir Area:		
Quabbin Park Cemetery	79.8	..
Relocation of Highways	229.7	6.9
Relocation of Transmission Lines	..	12.6
Miscellaneous	295.9	..
Miscellaneous in Ware Watershed	0.3	..
Total	1979.5	95.4

PART III.

DESCRIPTION OF THE PRINCIPAL CONSTRUCTION ITEMS.

Design and Construction of the Quabbin Aqueduct Tunnel.

The construction shafts were generally located on a side hill overlooking an area on which spoil from the tunnel excavations could be dumped and so that the dump cars could be carried along the top of the spoil banks on a slight down-grade and returned empty on the slight upgrade. The contractor's camp buildings as well as his construction camp were also located close to each shaft head. Particular attention was paid to the location of his buildings and the spoil banks at shafts 1, 2, 3 and 4 which were located within the watershed of Wachusett Reservoir. At each of these points the contractor was required to collect the drainage from the entire area in a settling basin and chlorinate the effluent. At Shaft 2, which was close to the Quinapoxet River, a main tributary of the reservoir, the drainage was also required to be filtered through slow sand-filter beds. At Shaft 1, close to the reservoir, plans were made for such filters, but the small amount of tunnel excavation done at this point and the small quantity of drainage involved made it unnecessary to construct them since the bottom of the settling basin acted as a natural filter.

The construction shafts are 14 ft. in diameter and were lined with

concrete as they were sunk to avoid the hazard of falling rock. This size of shaft allowed the installation of double hoists for tunnel excavation with space for wiring, water and compressed-air lines, ventilation pipe and a ladder for emergency exit. This 14-ft. diameter also limited the length of the base line dropped from the surface to determine the direction of the tunnel excavation. At Shaft 3 it was necessary first to sink a concrete caisson through 61 ft. of earth cover to reach sound ledge. At the other shafts ledge was located at depths varying from 3 to 34 ft. and ledge was uncovered by open cut and sheeted excavation of the earth. Each shaft, and in no case less than 135 ft. of the tunnel on either side, generally 200 ft., were excavated by bucket hoist. As soon as the blasting operations were at a safe distance from the foot of the shaft a timber head frame was erected over the shaft and the shaft timbered to support guides for the shaft hoists and other shaft equipment. On the Wachusett-Coldbrook section the skips loaded with tunnel spoil were run onto the hoist platform at the bottom of the shaft, brought to the surface and run in trains to the edge of the spoil bank, dumped and returned by the same route to the tunnel heading. On the Coldbrook-Swift section balanced skips were used in the shafts. The muck was dumped from the tunnel cars into the shaft skips or buckets which dropped into a recess below grade. At the surface, these buckets could be either automatically dumped at the top of the lift into muck cars or crusher bin or kept on an even keel and used to carry men, tools and equipment.

The rock encountered was a compact schist, varying to granite. The author has asked Frank E. Fahlquist, Assistant Engineer in the field organization, to contribute a brief discussion of the geology of the region from his intimate knowledge and experience in the construction. No attempt at any further description of the rock encountered will be made but reference should be had to Mr. Fahlquist's discussion. The rock was uniformly sound except for a few short stretches near the west end. A length of only 1 971 ft. required permanent timber support, which is only 1.52 per cent. of the entire length. In addition, about 450 ft. in scattered locations required temporary timbering. Also in several places, aggregating about 12 050 ft. the seams of the rock tended to disintegrate on exposure to the air so that frequent scaling was necessary to insure the safety of workmen. In such cases the stability of the roof was secured, pending the final concrete lining, by a coating of Gunite usually applied in three courses of about 1-in. total thickness.

At the east or Wachusett end, the borings indicated the necessity of lowering the grade of the tunnel to insure its construction in sound ledge with a sufficient cover to make the roof self-supporting during construction and to insure reasonable water tightness during operation. On account of the west to east slope of the tunnel, drainage during construction was easily taken care of in all the headings that extended westward from the shafts. In each of the easterly headings the grade of the tunnel, except

near the Wachusett end, extends level from the shaft, in order to facilitate the drainage operations and, in general, the grade dips down at the halfway point between shafts. At the Wachusett end, commencing near Shaft 3, to secure the rock cover as previously noted, the grade dips down steeply to a depth of 150 ft. at Shaft 2 and continues on the down grade to a depth of 200 ft. below the normal grade line at Shaft 1.

This tunnel should be classed as a grade tunnel, not a pressure tunnel. Although in operation it will be 100 ft. or more below the hydraulic gradient, nevertheless, the internal water pressure will be less than the external pressure due to the normal ground-water level. This will not be strictly true for a short distance at the Wachusett end where a length of 500 ft. next to Shaft 1 was constructed as a circular pressure tunnel with the lining grouted against the ledge to prevent leakage from the tunnel under the full head of Quabbin Reservoir, which, as previously noted, will be effective through the entire tunnel as far as the control gates at the top of Shaft 1. Another small section 200 ft. long at the extreme westerly end next to Shaft 12 will also be similarly constructed as a circular pressure tunnel in order to prevent excessive leakage from Quabbin Reservoir into the tunnel whenever it may be desired to unwater the tunnel by inserting stop logs at Shaft 12. With these two exceptions, the horseshoe shape is maintained throughout the entire length of the tunnel. The sides are vertical, 11 ft. 0 in. face to face, and the height from invert to arch is 12 ft. 9 in., the cross sectional area being the same as that of the circular sections, 12 ft. 9 in. in diameter.

The tunnel is lined with concrete for smoothness to increase the carrying capacity as well as to prevent disintegration of the rock and to insure a clean unobstructed tube, and excessive external pressures are prevented by frequent vents through the lining. The carrying capa city of the tunnel was found to correspond with a coefficient of Kutter's $n = .012$, and it is expected that this will increase with age to at least .013 but to not more than .014.

Excavation of the Tunnel. The full face method of driving the tunnel was used for most of the excavation at shafts 2 and 4 in the Wachusett-Coldbrook section and at all shafts in the Coldbrook-Swift section. At shafts 5, 6, 7 and 8 in the Wachusett-Coldbrook section, where the heading and bench method was used, the progress was materially slower. In drilling for excavation by the full face method a crew consisted of one foreman, 5 drillers, 5 driller's helpers and 2 steel nippers, using 5 drills mounted on a drill jumbo platform. As the work progressed in the Coldbrook-Swift section the number was reduced from 5 to 4. The position and number of holes varied with the condition of the heading and the kind of rock encountered. Ordinarily about 35 to 40 holes were drilled with delayed timing in the following groups:

	Shafts 2 and 4.		Shafts 9, 10, 11 and 12.	
	Number.	Depth Ft.	Number.	Depth Ft.
"Cut" holes — forming a central cone..	5	10	9	11½ to 12
"Line" holes — defining the break along sides and arch.....	13	8	15	9½ to 10
"Lifter" holes — to throw the muck up and back from the headings.....	4	10	9	9½ to 10
"Reliever" holes — to break up the rock between the "cut" and "line" holes	13	8½	7	9½ to 10

This drilling operation usually required about 3¾ hours. Loading started immediately after the holes were blown clean of dirt and water. At shafts 2 and 4, about 8 sticks of 40- to 60-per cent. dynamite, averaging

FIG. 5.—TUNNEL MUCKING MACHINE IN QUABBIN AQUEDUCT. JANUARY 20, 1930.

about 1.1 lb. per stick, were put into each hole making about 310 lb. per round. At shafts 9, 10, 11 and 12, from 10 to 14 sticks of 40- to 60-per cent. dynamite, averaging about 0.6 lb. per stick, were put into each hole making about 275 lb. per round. Drilling equipment was then taken back to a switch about 1 000 ft. from the heading; lights and air- and water-connections were taken back a safe distance and all the men left the heading which was shot from a switch located about 1 500 to 2 000 ft. from the heading. The net progress varied from about 6 to 8 ft. per round at shafts 2 and 4 and from about 7 to 9 ft. at shafts 9, 10, 11 and 12. The quantity of dynamite used per cubic yard of rock averaged about 6.45 lb. in the Wachusett-Coldbrook section, and about 4.7 lb. in the Coldbrook-Swift section, varying from 3.8 to 5.0 lb., depending on whether the rock was soft or fairly hard. The smaller amount of dynamite used per cubic yard in the westerly section

was due both to the difference in the dynamite and to soft rock encountered, particularly at Shaft 9. Approximately, 2 600 tons of dynamite were used in excavating the entire tunnel.

After the blast, the mucking crew consisted generally of eight men. Air-driven shovel-type mucking machines, capable of working in close quarters, were used at shafts 5, 6, 7 and 8 and electric-driven muckers at shafts 2, 4, 9, 10 and 11. In the east heading from Shaft 12, a scraper mucker was employed. At each shaft about three trains of 4 to 7 muck cars each were required, the number depending on the length of heading. For ventilation, air was exhausted through a 14-in. sheet-metal pipe in the shaft, and along the tunnel within about 200 ft. of the heading generally through spirally wound wood-stave pipe 14 in. in diameter. This wood pipe stood up very well in the face of occasional flying rock at this distance from the heading. Usually the air line to the heading was left open during a shot and until the heading was cleared of enough powder smoke for the mucking crew to start work. This hastened clearing the heading. While drilling, the exhaust from the drills furnished ventilation at the face of the heading.

To keep the drill crew on line, the direction of the tunnel line was transferred from surface monuments by dropping two plumb lines down the shaft. Of course these could not be much over 13 ft. apart. The line was dropped a number of times, when a Sunday or holiday gave opportunity, and projected into the tunnel where it was read on brass vernier scales set into the rock roof. Table 5 shows the accuracy obtained:

TABLE 5.— DIFFERENCES IN LINE AND GRADE AT THE POINTS WHERE THE EXCAVATION MET BETWEEN SHAFTS.

Shaft.	Distance Apart — Ft.	Difference in Feet		Point of Meeting.
		Line.	Grade.	
1– 2	9 902	0.33	0.01	836 ft. from Shaft 1
2– 3	9 167	1.85	0.02	8 537 ft. from Shaft 2
3– 4	8 819	0.81	0.04	1 481 ft. from Shaft 3
4– 5	11 633	0.24	0.02	5 814 ft. from Shaft 4
5– 6	11 102	0.49	0.13	6 329 ft. from Shaft 5
6– 7	9 232	0.31	0.07	4 732 ft. from Shaft 6
7– 8	10 048	0.09	0.14	5 545 ft. from Shaft 7
8– 9	12 412	0.06	0.06	4 998 ft. from Shaft 8
9–10	14 501	0.63	0.11	7 611 ft. from Shaft 9
10–11	15 840	0.12	0.02	8 278 ft. from Shaft 10
11–12	16 757	0.10	0.05	8 254 ft. from Shaft 11
12–Portal	319	0.00	0.02	230 ft. from Shaft 12
Average		0.42	0.06	

The typical contractor's plant at each shaft during the excavation operations consisted of 3 air compressors totalling about 1 800 c.f.m. capacity, one 100-h.p. drum hoist handling the cables through the shaft

head frame, one 60-h.p. blower with a capacity of about 4 000 c.f.m., 2 to 3 mucking machines, pumps, rolling stock, etc.

Progress made in sinking and lining the 14-ft. construction shafts and in excavating the tunnel is shown in the tables 6 and 7, which give the average and maximum progress made and the number of men employed by the contractor in these operations. The shallower construction shafts, 14 ft. in diameter, that is those about 200 to 275 ft. deep, were sunk and lined at the rate of about 25 to 30 ft. per week. At greater depths the progress was slowed up materially so that for shafts over 400 ft. deep the average rate of sinking and lining the entire depth was about 19 ft. per week. The contractor's force for the shaft-sinking operations averaged about 35 to 40 men at each shaft. After the tunnel headings were turned at the shaft bottoms and after head frames and hoisting equipment for the tunnel excavation were installed, a fair average progress of the excavation in each tunnel heading was, in round numbers, 100 ft. per week, although materially less at some of the shafts, as shown in the table. An excellent record was made at Shaft 9 during the week ending October 22, 1932, when 131 ft. were excavated in the east heading and 138 ft. in the west heading. The contractor's force for tunnel-excavating operations averaged about 75 to 100 men at each shaft. This number was later increased by about one-third during the subsequent tunnel-lining operations, working three shifts. The following comparisons are interesting:

In the Scituate tunnel of the Providence Water Supply, most of the excavation was done from a single shaft in 1923 and the first half of 1924. The average progress of excavation from this shaft per heading was about 65 ft. per week and the maximum 82 ft. This tunnel was somewhat smaller than Quabbin Aqueduct but large enough to use the smallest shovel-type of mucker. Practically no support of the rock roof was required.

The Shandaken tunnel of the New York Water Supply was excavated from 7 shafts in 1919–1923. The average progress of excavation per heading was about 65 ft. per week and the maximum about 610 ft. per month. The finished dimensions of this tunnel were 10 ft. 3 in. width by 11 ft. 6 in. height. It was a grade tunnel very similar to Quabbin Aqueduct. Forty-two per cent. of the total length required roof support.

The New York City Tunnel No. 2 was excavated from 16 of its 17 shafts in 1930 and the first half of 1931. The average progress of excavation per heading was about 70 ft. per week. Ordinarily 3 rounds or advances were made in two days in each heading. The maximum progress was made when working a single heading from one shaft. Under these conditions 2 advances per day were made, resulting in a maximum progress of 143 ft. per week. The finished diameter of this tunnel was 17 ft. About 26 per cent. of the total length required roof support.

Lining of the Tunnel. To obtain aggregates for the concrete tunnel lining, rock spoil from the tunnel excavation was crushed and screened at the top of each shaft from which the major lining operations were carried on.

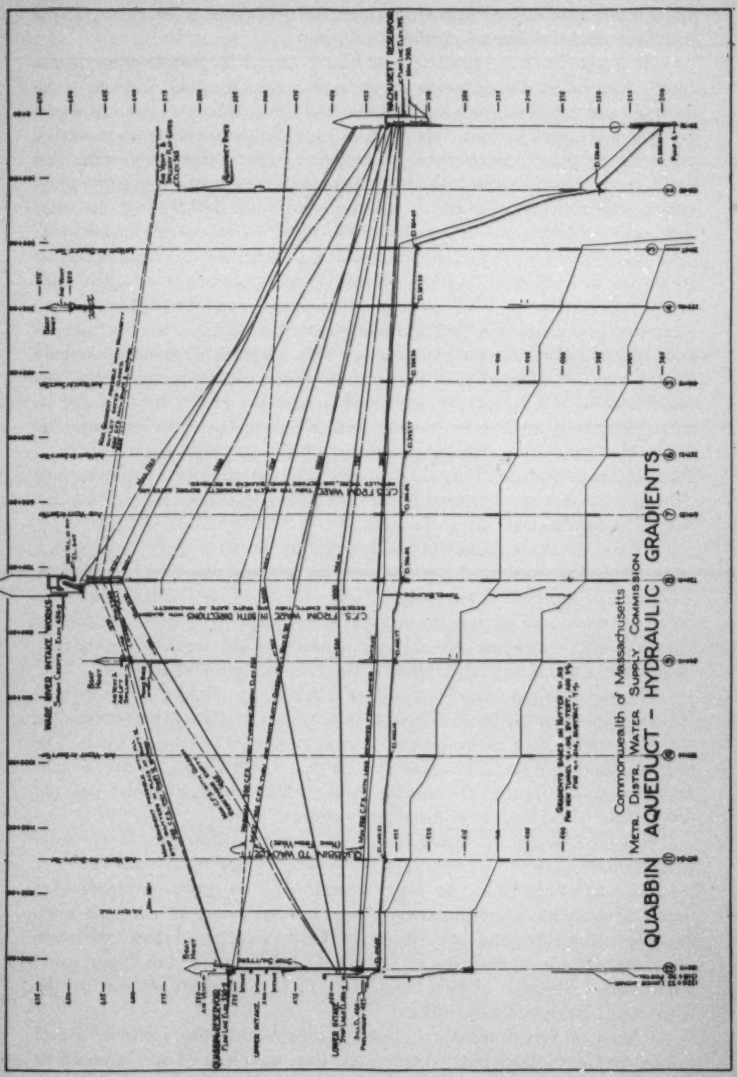

Fig. 6.— Hydraulic Gradients.

These were shafts 2, 4, 5, 7, 9, 11 and the portal. Lining is still in progress and practically completed at the latter point, and preparations are now being made for lining from Shaft 10. Also the short section of circular pressure tunnel at the east end of the aqueduct was lined from the mixing plant at the top of Shaft 1. Also concrete in the invert in the west heading from Shaft 8 was mixed at the top of this shaft and dropped in an 8-in. pipe embedded in the masonry of the foundations of the Ware River intake works while these works were under construction by another contractor. At several of the shafts a small shortage of fine aggregate was supplied by bank sand — at Shaft 4 from the tunnel spoil by the use of sand rolls. At Shaft 9 a considerable quantity of sand and gravel was borrowed, and prepared aggregates were hauled from Shaft 10 to supply a shortage of satisfactory tunnel spoil.

The Wachusett-Coldbrook section was lined from three tunnel mixing-and-placing plants, which were moved from point to point along the line of the work. These placed the side walls and arch only, leaving the invert to be placed in a separate operation. The aggregates were mixed at the top of the shaft, dropped to a bin at the bottom and loaded on trains for the mixing-and-placing plant where the water was added during mixing. The 1-cu. yd. mixer discharged into a concrete gun of the same capacity equipped with a piston 32 in. in diameter working through a 66-in. stroke and operated by air pressure of from 75 to 100 lb. per sq. in. This gun forced the concrete into a 6-in. chuting line about 125 ft. long extending over the top of the forms. Each tunnel plant used 200 ft. of collapsible steel forms. These forms were erected on wooden sills set to line and grade several hundred feet in advance of the work. Each 25-ft. section of forms was hinged for collapsing at three points, at the center of the arch and 30 in. from the bottom of each side wall. As each section of the form was collapsed, it was carried on a form jumbo through the interior of the sections remaining in place and again set up at the other end. The concrete was thus poured in one continuous operation, the chuting line discharging above the center of the arch and the wet concrete taking its natural slope down the two sides between the steel forms and the irregular rock excavation.

At the surface plants, the rock from the tunnel spoil banks was handled with a drag line scraper to the crushing and screening equipment, and the coarse and fine aggregates were stored in huge piles above timbered tunnels. In these tunnels the aggregates drawn from the storage piles above were weighed for each batch and on signal from the foot of the shaft, were released onto a belt conveyor, the proper amount of cement added and the dry batch conveyed to the top of the shaft where it was dropped through an 8-in. pipe to a receiving hopper at the foot of the shaft. The invert was placed later in a separate operation after the arch and side wall forms were out of the way. The concrete for this was mixed at the surface involving some rearrangement of plant at the shaft head.

The Coldbrook-Swift section is being lined in a somewhat different

manner, from four tunnel placing plants. The concrete is mixed at the surface, dropped down the shaft through an 8-in. pipe to a hopper at the bottom and carried to the placing plant in especially designed cars. Lining instead of being a continuous operation is poured in sections requiring end bulkheads. The forms, which are carried ahead on a jumbo in one operation, consist of 100 to 125 ft. of side-wall forms up to about 2 ft. below the springing line of the arch followed by the same length of arch forms, the entire jumbo, including a 60-ft. track incline, being supported on a wide-gage track accurately placed to line and grade. Under this arrangement concrete for the side walls is dumped directly from the cars and that for the arch is placed through a 6-in. chuting line from a concrete gun. To date this same gun has been used to discharge concrete through the arch forms for the placing of the invert which keeps pace with the side walls and arch, but the preparations for lining at Shaft 10 include plans to carry concrete for the invert in cars directly through the arch and side-wall forms in order to speed up the progress.

Progress made in lining the tunnel is shown in Table 8. A fair average progress of the tunnel-lining plant for placing side walls and arch was about 850 ft. per week, for the separate operation of placing the invert as was done in the Wachusett-Coldbrook section about 1 500 ft. per week, and for the complete lining operation as done in the Coldbrook-Swift section about 320 ft. per week in each of two headings worked from one shaft plant.

However, the method of lining used at shafts 9, 10 and 11 had the advantage of completing the entire tunnel section in one operation; also two tunnel plants or sets of forms, one in each heading, were supplied by the mixing plant at each of these shaft heads so that a fair average progress for the shaft concrete plant was 640 ft. per week, twice that obtained by each tunnel plant. Where forms of sufficient length are provided, it is the capacity of the concrete plant at the shaft head that limits the output in each case. The most recent progress, from the portal, was apparently limited to 536 ft. per week solely on account of the 100-ft. length of forms in use at that point. Plant now being installed at Shaft 10 includes two 2-cu. yd. mixers, instead of the 1-cu. yd. mixers previously used, and 125 ft. of forms in each heading.

In comparing the rates of progress made in the Wachusett-Coldbrook and Coldbrook-Swift sections it should be borne in mind that in the former case it was necessary to complete the entire required length of side walls and arch in each heading and then to finally grade and clean up the bottom before the work of placing invert could be started.

For comparison: the Shandaken tunnel of the New York Water Supply, which as previously stated was very similar to Quabbin Aqueduct, was lined from 5 of the 7 shafts. The maximum progress, arch, side walls and invert, was 2 936 ft. per month.

The amount of electric power used in the Quabbin Aqueduct tunnel construction is shown in Table 9 which indicates that throughout the period of excavation and before lining was started, the amount of energy

used for all purposes ran very close to 40 k.w.h. per cubic yard of rock excavated.

Tunnel Measurements for Payment. On all contracts for shafts and tunnel the contractors were paid at unit bid prices computed on the basis of fixed thickness of lining. For each cross-section a so-called "A" line was established as the line beyond which no points of rock were allowed to project. This then defined the minimum thickness of the concrete lining. In sound rock this was fixed at 3 in. on the side walls and invert and 8 in. on the

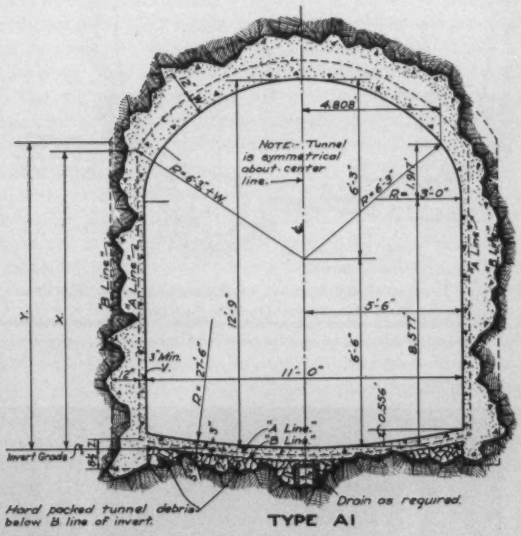

FIG. 7.— TYPICAL CROSS SECTION OF QUABBIN AQUEDUCT TUNNEL.

arch. There was also a "B" line which, except in certain cases where permanent timber support was required, was always a fixed distance from the "A" line, namely 12 in. on the side walls and arch and 2 in. on the invert. The "B" line was generally the pay line for both excavation and concrete, and was intended to represent a fair average of the irregular rock breakage. The actual breakage of rock beyond the "A" line for the side walls and arch averaged 11⅜ in. on the easterly portion of the Wachusett-Coldbrook section, 9¾ in. on the westerly portion of this section and 10⅝ in. on the Coldbrook-Swift section, or a weighted average for the entire tunnel of a little over 10½ in. This means that the actual quantities of rock excavated and of concrete lining placed were respectively 2.6 and 8.4 per cent. less

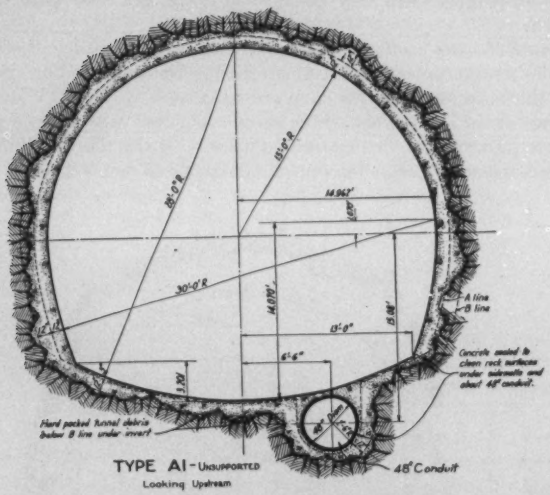

FIG. 8.—TYPICAL CROSS SECTION OF RIVER DIVERSION TUNNEL. Main Dam Site — Quabbin Reservoir.

FIG. 9.—COLLAPSIBLE AND TELESCOPING STEEL FORMS FOR CONCRETE LINING OF SIDE WALLS AND ARCH, QUABBIN AQUEDUCT TUNNEL. NOVEMBER 4, 1930.

than the theoretical or pay quantities reported in the tabular information in Part IV. Except at occasional sections where the concrete invert was carried down to sound rock for a water cut off, the rock débris on the floor of the tunnel was merely graded to a smooth bed, at the required "B" line, on which the concrete was placed. In addition a "C" line so-called was established as the pay line in certain sections where the "A" and "B" lines did not apply on account of the irregular shape of the volumes required to be excavated.

The contractor continually strove to obtain the most economical balance between the various items of the work. If the original drilling and blasting did not break the rock outside the required line and grade, expensive trimming was required at the projecting points. On the other hand if the original breakage was excessive the concrete lining had to be that much thicker. The engineers, following the blasting operations, painted with white paint all points left projecting beyond the "A" line. Enlarging the rock section by additional drilling and blasting was of course expensive but, on the other hand, if too large a section was broken out, it had to be filled with concrete later at the contractor's expense. The distance of 12 in. between the "A" line and the "B" or pay line, for both excavation and lining was determined in advance from experience on other work of this character and is believed to have worked out very satisfactorily in securing fair bidding and adequate compensation to the contractor.

Shaft Head Houses. Upon the completion of the tunnel excavation and lining, the construction shafts were capped in such a way that they can be reopened and used for access at any time in the future in case major construction or repair operations should become necessary. The cover consists of seven removable reinforced concrete slab sections resting on the top of the shaft lining and on two removable beams. This slab is covered by 1½ ft. of earth. An air vent protected by a locked steel grating is built into one side of the shaft at the top and is so designed as to prevent the committing of nuisances and malicious mischief.

A special construction was required at the top of Shaft 2 where four heavy hinged covers were placed in the shaft cap to act as spillway flap gates, air venting being taken care of by four breather pipes. The whole is surrounded by a high steel fence to prevent tampering. The need for a spillway at this point is presented by the very remote but not impossible contingency of the control gates at Shaft 1 being closed at a time when flood flows from the Ware River at Shaft 8 are being discharged into the tunnel at a very high rate. Under such conditions this spillway would reduce the static head tending to lift the top of Shaft 1. At Shaft 2 there is also the possibility of diverting the Quinapoxet River into the tunnel. This river is one of the principal tributaries of Wachusett Reservoir and such a diversion would not increase the drainage area of the supply but would make it possible to store Quinapoxet water in the higher Quabbin Reservoir. This would be particularly desirable if Wachusett Reservoir were well filled at a time

TABLE 6.— QUABBIN AQUEDUCT, PROGRESS IN SINKING AND LINING 14-FOOT CONSTRUCTION SHAFTS.

Shaft No.	1*	2	3	4	5	6	7	8*	9	10	11	12*
Total depth — ft.	221	313	339	457	656	432	411	253	200	408	275	90
Depth of earth cover — ft.	30	25	61	23	19	29	25	3	9	34	24	4
Average weekly progress of excavation — ft.:												
Year ending Nov. 30, 1927	....	21.1	19.2	19.2	19.8	16.5	18.6	....	....	....	....	....
Year ending Nov. 30, 1928	11	....	....	....	....	....	....	13.1	....	....	....	....
Year ending Nov. 30, 1931	....	....	....	....	....	....	....	....	23.5	18.5	33.7	7.3
Maximum depth of rock excavated in one week — ft.	20	41	40	31	47	35	35	25	33	43	44	19
Average number of men employed during typical period, in two shifts:												
1927	....	35	24	37	39	31	35	....	....	....	....	....
1931	....	....	....	....	....	....	....	....	41	37	37	....

* Larger Waterway Shafts Involving Special Construction.

TABLE 7.—QUABBIN AQUEDUCT, TUNNEL EXCAVATION PROGRESS.

From Shaft No.	1	2	3	4	5	6	7	8	9	10	11	12	Portal
Excavated under shaft contracts—ft.													
Year ending Nov. 30, 1927	....	443	.....	.....	.....	.....	251	.....	.....	.....	.....	.....	....
Year ending Nov. 30, 1928	....	1 328	1 196	843	1 367	1 340	1 471	.....	.....	.....	.....	.....	....
Excavated under tunnel contracts—ft.													
Year ending Nov. 30, 1928	....	1 760	.....	888	1 956	2 177	2 038	278*	.....	.....	.....	.....	....
Year ending Nov. 30, 1929	836*	9 614	.....	8 883	8 463	5 988	5 738	7 920	.....	.....	.....	.....	....
Year ending Nov. 30, 1930	....	5 373	.....	2 538	362	.....	547	1 303	.....	.....	.....	.....	....
Year ending Nov. 30, 1931	....	.....	.....	.....	.....	.....	.....	.....	1 149	44*	846†	573†	....
Year ending Nov. 30, 1932	....	.....	.....	.....	.....	.....	.....	.....	12 116	7 563	11 190	4 929	89
Year ending Nov. 30, 1933	....	.....	.....	.....	.....	.....	.....	.....	1 760	7 561	3 780	3 231	....
Total	836	18 518	1 196	13 152	12 148	9 505	10 045	9 501	15 025	15 168	15 816	8 733	89
Average weekly progress of excavation per heading, after head frames were placed — ft.													
Year ending Nov. 30, 1928	....	70	.....	63	80	63	63	.....	.....	.....	.....	.....	....
Year ending Nov. 30, 1929	....	100	.....	85	83	74	58	75	.....	.....	.....	.....	....
Year ending Nov. 30, 1930	....	101	.....	103	78	.....	49	74	.....	.....	.....	.....	....
Year ending Nov. 30, 1931	....	.....	.....	.....	.....	.....	.....	.....	68	.....	.....	.....	....
Year ending Nov. 30, 1932	....	.....	.....	.....	.....	.....	.....	.....	115	92	107	93	....
Year ending Nov. 30, 1933	....	.....	.....	.....	.....	.....	.....	.....	116	93	94	113	....
Maximum progress in one week, two headings in Shaft 2 to 11 inclusive, one heading in Shaft 12 — ft.	....	232	.....	237	192	187	142	.....	269	241	250	136	....
Length requiring permanent timbering — ft.	....	35	324	.....	.....	.....	22	.....	161	1 058	252	119	....
Length requiring Gunite treatment for temporary support	....	.....	.....	2 615	1 825	.....	.....	.....	3 950	3 660	.....	.....	....
Average number of men employed during typical period, in two shifts:													
1929	35	119	.....	125	90	84	67	79	.....	.....	.....	.....	....
1932	....	.....	.....	.....	.....	.....	.....	.....	78	64	74	58	....

* All before head frame was placed.

† Practically all before head frame was placed.

when Quabbin Reservoir was depleted. Such an arrangement would also increase the electric energy capable of being developed at Shaft 1. The existing legislation already provides for such a diversion and a recess has been excavated in the rock opposite a break in the concrete lining of Shaft 2 at a point where such a future connection into the shaft may be made.

Other special constructions were at shafts 4 and 9. At Shaft 4 the water would be backed up from Wachusett Reservoir to a depth varying

FIG. 10. — CONSTRUCTION OF SIPHON SPILLWAYS IN WARE RIVER INTAKE WORKS.
JULY 2, 1930.

from 2.7 to 7.7 ft. in the 12 ft. 9 in. tunnel, and this is the most easterly shaft which could be reached for inspection purposes by a boat in the tunnel without resorting to pumping. A head house was constructed at the top of this shaft, including provision for attaching a cable hoist to the roof frame so that a boat can be lifted out of the tunnel. This shaft is about 460 ft. deep. No ladder is provided but access is obtained only by bucket hoist. The tunnel at this point is divided by a central pier which supports a platform a few feet above the tunnel arch. This platform provides a landing place for the bucket and also operating space for placing and removing stop logs in the two bays of 4 ft. 9 in. clear span on either side of the central pier. In case it should be necessary for inspection purposes to interrupt the flow in the tunnel temporarily at this point such stop logs would be capable of backing the water up in the tunnel as far as Shaft 10. Permanent hoisting equipment has not yet been installed in the head house. The hoist used to date is mounted on an automobile truck body and is used at other shafts and

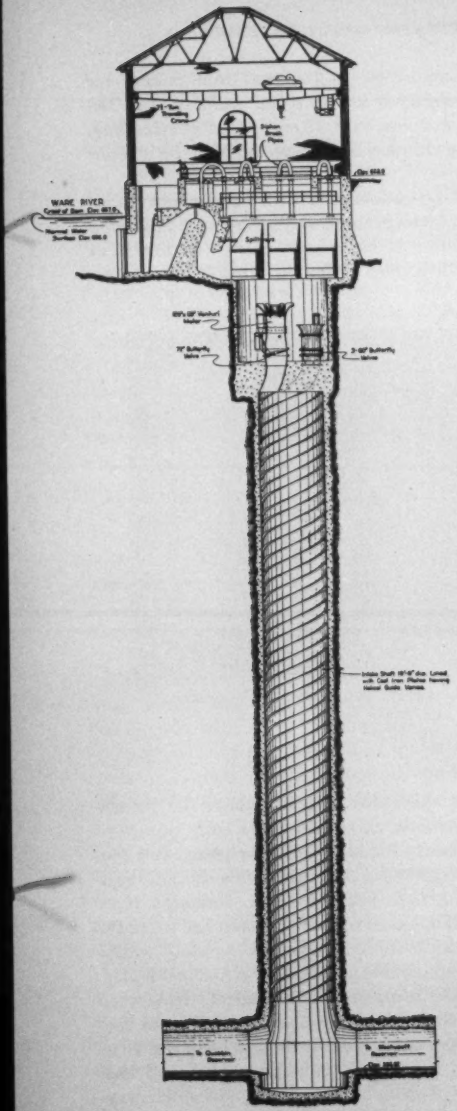

FIG. 11.

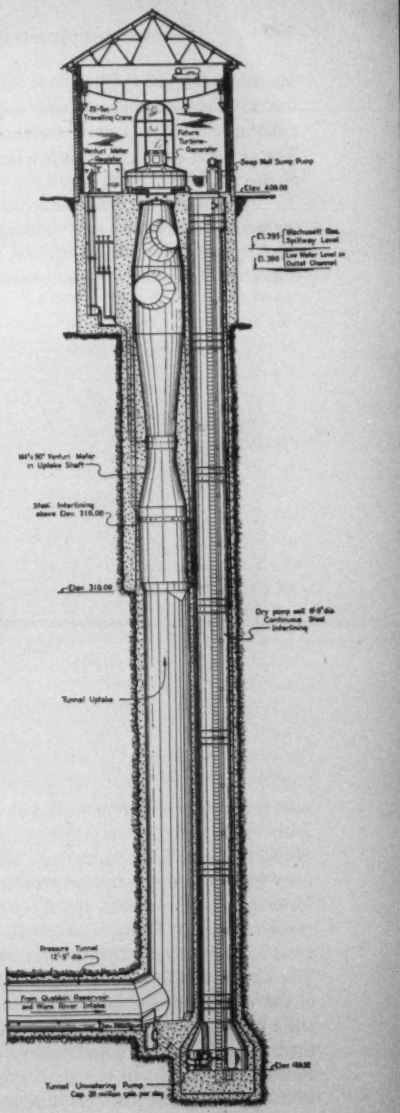
25-Ton Travelling Crane
Venturi Meter Chamber
Future Turbine-Generator
Deep Well Sump Pump
Elev. 409.00
El. 395 Wachusett Res. Spillway Level
El. 390 Low Water Level in Outlet Channel
144"x 90" Venturi Meter in Uptake Shaft
Steel Intarlining above Elev. 316.00
Elev. 319.00
Dry pump well 6'-0" dia. Continuous Steel Intarlining
Tunnel Uptake
Pressure Tunnel 12'-0" dia.
From Quabbin Reservoir and Ware River Intake
Elev. 169.32
Tunnel Unwatering Pump Cap. 20 million gals. per day

FIG. 12.

Ware River Intake Works at Shaft 8. Wachusett Outlet Works at Shaft 1.
QUABBIN AQUEDUCT TUNNEL.

for other purposes. However, a special cable on a special drum is kept for use in the tunnel shafts and is employed for no other purpose, and the automobile hoist is used for this purpose only after special overhauling. The air vent at this shaft is in a false chimney not connected with the interior of the house.

At Shaft 9, a head house will be constructed with similar provisions for bucket and boat hoist, landing platform above the tunnel arch, and stop logs capable of backing up the water in the tunnel above the invert at Shaft 12. A further provision is made at Shaft 9 for the possible installation

FIG. 13.—CONSTRUCTION OF VALVE PIT AND HELICAL CAST-IRON LINING OF SHAFT 8, WARE RIVER INTAKE WORKS. AUGUST 19, 1930.

of a pump in a chamber in the rock wall at the platform above the tunnel arch which could, if required, supplement the tunnel unwatering pump at Shaft 1. There is no immediate necessity for such pumping plant, and the only provision made in the present construction program is a suction well formed in the concrete lining of the tunnel and a 16-in. discharge pipe embedded in the lining of the shaft. This shaft is not very deep and a 210 ft. steel ladder with platforms every 31.2 ft. will be attached to the shaft walls. The air vent at this shaft will be taken off the side of the top independently of the interior of the head house in the same way as at Shaft 4. However, the top of this shaft is not very much above the hydraulic gradient in the tunnel under certain conditions of operation. The water level will rise to a considerable height in the shaft even when no air is being vented, and the venting of air under such conditions will result in an air lift action which may cause a foamy mixture of air and water to fill the entire top of the shaft. Accordingly, the air vent here is also designed as a spillway with an outlet

to Prince River, an adjacent stream, so that foam and if necessary water can be discharged at this point. In addition to shafts 4 and 9, access to the tunnel by boat will also be provided at the Swift River intake in the head house to be constructed at Shaft 12.

Importance of Air Venting. Although as previously stated Quabbin Aqueduct is not a pressure tunnel in the strict sense of the word, it is precisely that so far as the problem of air accumulations and venting is concerned. Although often neglected, this problem is an important one to consider in the design of any conduit whether a large tunnel or a small pipe. Some interesting phases of the problem as it occurs in a small pipe have already been referred to in the case of the southern Sudbury pipe lines. In large tunnels under high heads, accumulations of air are capable of venting themselves with destructive force. The Ware River intake works are designed to prevent the entrance of air along with the flood flows that are dropped down Shaft 8. On this account, as soon as the water in the tunnel rises sufficiently to seal the bottom of the shaft, the water rises in the shaft about 25 ft. above the hydraulic grade, under a vacuum of about 22 in. of mercury. In spite of this, considerable air does get into the tunnel with the Ware floods and probably more is sucked out of the water by the vacuum effect. This air gradually rises and travels along the top of the tunnel. In order to facilitate travel of this air away from Shaft 8 the tunnel is made level the entire distance to Shaft 7. Any air not vented at Shaft 7 finds vents at five other shafts before reaching the control gates at Shaft 1. Similarly a discharge westward from Shaft 8 finds air vents at each shaft.

The problem is not entirely solved however by finding vents for continuous air flow in the direction of water flow. A much more destructive venting of air can be caused merely by a sudden change in the rate of flow. Mention has already been made of the change of grade or slight hump in the tunnel halfway between two adjoining construction shafts. The distance between shafts is so great that under certain conditions of low velocity an air bubble of large volume could accumulate in the top of the tunnel even on a straight grade and where these humps exist this accumulation may be intensified. This will happen regardless of the elevation of the hydraulic grade, which may be over 150 ft. above the tunnel corresponding to an air pressure of 70 lb. per sq. in. Sudden closure of the gate, for example at Shaft 1, would cause the air which accumulated gradually to be vented suddenly. This could occur at any of the shafts, but most of them are of such height that they each have a very large volume of air cushion to absorb the shock. As an additional safeguard, the access covers in the floors of the head houses at shafts 4 and 9 are hinged at one side and fastened at the other with specially designed clamps, and a fixed louver for ventilation is provided in one window.

In the head house at Shaft 12, however, it would be impractical, on account of provisions to be made for handling stop shutters, screens, etc., and access to the basement, to fasten the floor covers. Hence the tunnel

TABLE 8.—QUABBIN AQUEDUCT, TUNNEL LINING PROGRESS, IN FEET (APR. 21, 1934).

Aggregate from Crushing and Screening Plant at	Side Walls and Arch from Tunnel Mixing and Placing Plants, Aggregates Batched at Shaft Head.		Near Shaft Bottom, Concrete Mixed at Shaft Head.	Invert as a Separate Operation, Concrete Mixed at Shaft Head.		Side Walls, Arch and Invert from Tunnel Placing Plant, Concrete Mixed at Shaft Head.		Average Number of Men Employed at the Shaft Head During Typical Period.	
	Total Concreted in One Stretch.	Average Per Week.		Total Concreted in One Stretch.	Average Per Week.	Total Concreted in One Stretch.	Average Per Week.	Two Tunnel Plants.	One Tunnel Plant.
Shaft 1	.....	....	526	526	....	.....	....	....	....
Shaft 2	9 054	900	241	8 504	1 300	.....	....	193	....
	9 237	862	....	9 365	1 060	.....	....	....	....
	.....	....	....	27	....	.....	....	....	....
Shaft 4	2 831	991	177	5 354	1 780	.....	....	128	....
	3 269	693	....	9 380	1 360	.....	....	....	....
	360	840	....	352	820	.....	....	....	....
	4 752	875	....	207	720	.....	....	....	....
	5 075	772	....	.....	....	.....	....	....	....
Shaft 5	11 079	835	34	11 024	800	.....	....	....	70
	3 977	994	....	5 782	1 850	.....	....	....	....
	.....	....	....	38	....	.....	....	....	....
Shaft 6	.....	....	22	.....	....	.....	....	....	....
Shaft 7	8 307	765	150	8 857	1 940	.....	....	....	96
	4 951	....	....	126	....	.....	....	....	....
	9 952	876	....	10 022	1 280	.....	....	....	....
	845	....	....	302	710	.....	....	....	....
	.....	....	....	22	....	.....	....	....	....
Shaft 8	.....	....	31	4 951	870	.....	....	....	....
	.....	....	....	31	....	.....	....	....	....
Shaft 9	.....	....	....	.....	....	7 419	322	104	....
	.....	....	....	.....	....	6 193	344	....	....
Remaining to be lined.	.....	....	....	.....	....	41	....	....	....
Shaft 10	.....	....	....	.....	....	.....	....	....	....
Remaining to be lined.	.....	....	....	.....	....	16 833	....	....	....
Shaft 11	.....	....	....	.....	....	7 284	270	109	....
	.....	....	....	.....	....	7 941	331	....	....
Remaining to be lined.	.....	....	....	.....	....	27	....	....	....
Shaft 12 and Portal	....	....	....	.....	....	8 580	536	....	67
Remaining to be lined.	.....	....	....	.....	....	538	....	....	....
Totals	73 689	....	1 181	74 870	....	54 862	....	....	....

TABLE 9.— QUABBIN AQUEDUCT, ELECTRIC POWER USED ANNUALLY BY SHAFT AND TUNNEL CONTRACTORS FOR PUMPING, EXCAVATING AND LINING EQUIPMENT, ETC.

Expressed in Thousand Kilowatt Hours.

The figures for Shafts 1–8, inclusive, include transformer losses. Those for Shafts 9–12, inclusive, exclude transformer losses — of 5 to 6 per cent.

Shaft.	Period of Rock Excavation.	YEAR ENDING NOVEMBER 30							
		1927	1928	1929	1930	1931	1932	1933	1934*
1	Aug. '28–June '29	0	83	320	308	136	.....	.....	.....
2	July '27–June '30	137	848	2,971	3,597	597	.....	.....	.....
3	Aug. '27–May '28	96	546	17	0	0	.....	.....	.....
4	July '27–Apr. '30	185	698	2 957	2 918	616	.....	.....	.....
5	May '27–Jan. '30	273	965	2 298	1 837	231	.....	.....	.....
6	July '27–Sept. '29	102	680	1 641	984	7	.....	.....	.....
7	June '27–Feb. '30	156	878	2 652	2 404	394	.....	.....	.....
8	June '28–Feb. '30	0	247	2 730	3 293	324	.....	.....	.....
9	July '31–Jan. '33	.....	.....	.....	.....	314	2 167	1 330	83
10	Aug. '31–Oct. '33	.....	.....	.....	.....	344	2 719	3 085	10
11	July '31–Apr. '33	.....	.....	.....	.....	307	2 577	3 104	523
12(Portal)	Aug. '31–June '33	.....	.....	.....	.....	322	1 636	1 142	576
Total		949	4 945	15 586	15 341	3 592	9 099	8 661	1 192
Total Rock Excavation — cu. yd.		24 000	120 400	324 700	72 600	25 500	231 800	121 300	None

* 4 months to April 1, when 19 042 ft. remained to be lined.

must be vented freely through the interior of the building to fixed window louvers. There will be no high velocity or drop in the level through the Swift River intake works, and in ordinary operation very little air can find its way into the tunnel in the vicinity of Shaft 12. Furthermore to prevent any dangerous accumulations of the air at this point a vent hole at the end of the level stretch east of Shaft 12 has been constructed extending from the tunnel arch to the surface of the ground. This vent was made by drilling an 8-in. hole 152 ft. deep to the tunnel arch and grouting in a 6-in. copper tube with a suitable vent head at the surface.

Temporary Tunnel Bulkhead. Upon completion of the Wachusett-Coldbrook section a massive steel bulkhead was constructed at a point 2 200 ft. west of the Ware River intake works at Shaft 8 capable of withstanding a working pressure of at least 100 lb. per sq. in. The bulkhead was designed to allow the use of the completed easterly section of the tunnel without interruption of the construction work on the tunnel extension to the Swift River. This bulkhead will also allow diversions from the Ware to continue until completion of Quabbin Reservoir to the point where it can store the Ware floods. In order to allow a flow in the eastward direction through the bulkhead under any emergency conditions, particularly to make possible an emergency diversion from a portion of the Swift River watershed in the event of a water shortage, this bulkhead contains two massive swinging doors with specially designed sensitive hinges and is in effect a huge check value.

Tunnel Unwatering Plant — Wachusett Outlet Works. Mention has already been made of the fact that the only tunnel excavation at Shaft 1 was a short length excavated by bucket hoist only, and that no head frame was required at this shaft. In order to provide more time for the construction of the special works required in and at the top of Shaft 1, the contractor pushed the work at Shaft 2 as rapidly as possible and as good progress was made in both headings and no unsound rock or other difficulties were encountered no further excavation from shafts 1 and 3 was required, and no head frames were erected. The west heading from Shaft 2 was holed through to Shaft 3, carried past the bottom of Shaft 3 and holed through to Shaft 4.

The cross section of Shaft 1 was oval in shape, 25 ft. long and 16 ft. wide, inside the first or temporary concrete lining which served for all tunnel excavating and lining requirements. The completed shaft contains the uptake or waterway shaft in the large end of the oval, and in the small end a dry well with a pump chamber at the bottom for use in unwatering the tunnel which, as already noted, was required to be depressed for a length of 3.50 miles at the easterly end in order to ensure a sound rock roof. This dry well was made tight by a continuous steel plate lining. The upper 95 ft. of the waterway shaft also required a steel lining thoroughly embedded in sound ledge for a height of about 65 ft. This was done to prevent the top of the shaft being blown off under the high head that will later be effective due to the elevation of Quabbin Reservoir when control gates are

installed at this point. Near the top of the waterway shaft a Venturi meter was molded in the concrete and mortar lining of this steel plate reinforcement. This Venturi meter is 144 in. by 90 in., and access to the tunnel can be had in an emergency by removing the shaft cap and lowering a bucket through the 90-in. throat.

Above the meter there are two horizontal branches, 7.5 ft. in diameter, on which control gates will be placed, one of these is a waste gate discharging below the level of Wachusett Reservoir, the other is provided as a connection to a penstock for a water wheel which, after the completion of Quabbin Reservoir, can generate power between the Quabbin Reservoir and Wachusett Reservoir levels, a total drop of 135 ft. Much of the total head will be used up in friction in the tunnel, and the maximum rate at which power can be economically generated is about 3 500 to 4 000 k.w. The building has now been left with an open well for such a future installation of power-generating equipment, and no control gates or shaft cap have yet been placed, the two 7.5-ft. openings being of ample capacity to discharge the maximum diversion of flood flows eastward from the Ware River without putting the shaft head under pressure. The Venturi meter also operates for the present with floats which will be replaced by high-pressure mercury chambers when the gates are installed and the full reservoir head is applied to the shaft.

The dry well is 8 ft. in diameter inside a 9-in. concrete lining of the riveted and welded steel shell. This enlarges to a 15-ft. diameter pump chamber at the bottom located 10 ft. below the invert of the tunnel. As the steel plate lining of this shaft was erected from the bottom up there were embedded in the concrete lining two cast-iron pipes, one 20-in. pipe for the pump discharge and one 16-in. pipe for ventilation to the electric motor. The only holes through the steel lining are for the 20-in. pump suction from a well from the bottom of the tunnel, for the 16-in. pump discharge, for the 16-in. motor vent pipe, and for two 1-in. holes for gage-glass connections so that when the tunnel is unwatered the water level can be watched and maintained by throttling the valve on the pump discharge at the bottom of the dry well. This welded steel lining encased in concrete proved to be practically bottle tight, although there is some moisture due to sweating on the concrete lining. A slight leakage drips in through the vent pipe, the bell and spigot joints of which are entirely outside the steel plate lining. A 20-in. guard gate of solid bronze occupies a strategic position between the bottom of the tunnel and the pump well and is secure from any falling objects. This gate is operated from a stand on the main floor by the pull on a continuous chain 418.5 ft. long. To provide for routine or emergency unwatering of this dry well there is installed on the main floor at the top of the well a pump of the deep-well type whose plungers operate in the bottom of a 6-in. steel casing hung in the well. A steel ladder with platforms every 33 ft. provides a more rapid access to the pump chamber at the bottom than the bucket hoist, although the latter is perhaps a more convenient means of exit than this 220-ft. climb.

The blower installed at the shaft head can supply 6 000 c.f.m. of air, which proved to be ample for the 900-h.p. induction motor driving the tunnel unwatering pump. The motor is stored on the main floor at the top of the shaft when not in use. Electricity is supplied at 220 volts through lead-covered cables hung in the well alongside the deep-well pump casing. The pump is capable of delivering 1 400 g.p.m. against a 205-ft. head and is permanently installed at the bottom of the shaft between the 20-in. suction and 16-in. discharge gates. The impeller shaft and the pump-half of the flexible coupling are of stainless steel, and the pump casing is kept filled with oil and inspected periodically. A small drain on the 16-in. discharge line provides a means for draining it in case of clogging of the pump or discharge valve during or after operation, and is ordinarily kept open to prevent any accumulation of head and possible leakage past the discharge valve into the oil-filled pump casing. The level of the oil in the casing is maintained a few inches below an open vent in the top of the casing, and a drain cock in the bottom of the casing maintains the necessary slow drip of water to provide for any very slight gradual leakage past the 20-in. bronze guard-gate on the pump suction.

The outlet to the reservoir is through three 8-ft. bays in which stop shutters can be placed to unwater the tunnel. From the top of the shaft the 20-in. pump discharge-pipe is embedded in the foundation masonry and extends to a point on the reservoir side of the shutter grooves. A travelling bridge crane in the building has a 20-ton hoist for heavy equipment, particularly the expected future generator rotor, and a 5-ton hoist for lighter uses in handling equipment in and out of the deep shaft wells. The travel of the bridge and of the 20-ton and 5-ton hoists on the bridge is effected manually, and the lift of the 5-ton hoist is electrically driven. The main-door frame of bronze is designed so that the side-light sections are removable, if necessary, for taking large pieces of equipment in and out of the building. All the steel roof-truss members are wrapped with wire fabric and coated with Gunite.

The Wachusett-Coldbrook section of the tunnel was unwatered in September, 1931, after the first season of its use for the diversion of Ware floods into Wachusett Reservoir. In spite of the fact that every care was exercised to thoroughly clean the 14-mile length of tunnel draining to the pump, it was considered inadvisable to place a screen over the inlet, during the first pumping operation, due to danger of clogging. A temporary steel rack was erected across the tunnel invert a short distance upstream from the suction well which caught any heavy débris to be removed after the unwatering. Considerable fine débris was removed from the pump casing itself. Several days were spent in warming up the motor and in miscellaneous preliminary work including cleaning out the pump casing. After pumping about 36 hours, the water level was pulled down until the lower end of the tunnel was about half full. The tunnel was then entered by boat lowered through the throat of the Venturi meter and a permanent screen of stainless

steel, a little over 2-in. mesh, was installed over the pump-suction well. In the meantime the tunnel had been inspected by two parties, one entering at Shaft 8 walking eastward and leaving by the bucket hoist at Shaft 4, a distance of 7.95 miles, another party entering at Shaft 4 and following the receding water level eastward and leaving through the Venturi meter at Shaft 1, a distance of 5.29 miles. The entire work was found in excellent condition. To facilitate inspection work, station locations in the tunnel are marked every 500 ft. and at shaft bottoms by 9-in. by 4-in. glazed tiles bolted to the walls.

Design and Construction of the Ware River Intake Works. At Shaft 8, for tunnel construction purposes, the first or preliminary lining was 20 ft. in diameter, and the tunnel contract included enough of the concrete building foundations to act as a cofferdam to prevent high water in the river from getting into the shaft. The tunnel contractor also placed the final shaft lining, which consisted of cast-iron segments 9 ft. 9 in. by 3 ft. 8 in. by $\frac{7}{8}$ in. with helical ribs which formed a sort of continuous 6-pitch screw thread projecting 6 to 10 in. into the waterway. There was no machine work on these castings, but they fitted together so neatly by means of lugs and sockets that they served as forms to hold the concrete backing without any calking of seams. This cast-iron lining was designed for the purpose of preventing destructive scour at the sides and bottom of the shaft due to the tremendous energy in this fall which is 50 per cent. higher than Niagara Falls. The valves at the top of the shaft discharge tangentially at a velocity of 30 to 40 ft. per sec. against the shaft walls, and without any great increase in velocity this lining maintains the circular motion to the bottom of the shaft so that the water is at all times pressed firmly against the walls by centrifugal force and has no opportunity to impinge directly against any surface. Before the final designs were made, this action was studied with the help of tests on a small model in the hydraulic laboratory of the Worcester Polytechnic Institute.

Among other pipes embedded in the concrete masonry back of the shaft lining, small pipes were installed to and from an air-lift fitting at a flowing seam encountered in the ledge during excavation, and by this means an excellent water supply for use in the intake building was obtained.

A separate contract was let for the diversion dam and the substructure of the building. The dam across the river is curved in arch form in order to increase the spillway length and also because this particular design enhanced the beauty of the entire structure. The spillway is 208 ft. long at elevation 657 above Boston City Base, 127 ft. above the flow line of Quabbin Reservoir and 262 ft. above that of Wachusett Reservoir. The diversion dam is about 17 ft. high, and this head is used solely for the operation of weirs and meter which discharge down the stream the required natural flow up to 85 m.g.d., and for emergency by-pass gates of greater capacity.

The diversion of flood flows above this specified limit is made through

a battery of 9 siphon spillways varying in capacity from 60 to 550 c.f.s. under a 9-ft. head. These siphons skim off the flood flows above the 85-m.g.d. rate and hold the level of the small diversion reservoir practically constant regardless of the rate of diversion. These siphons discharge into a central valve pit, the water level in which varies with the head required on the siphons. The water in the pit acts as a seal to prevent the entrance of air into the tunnel. In the bottom of the pit are four butterfly valves of the Dow-disc type which control the discharge down the shaft. They are operated by oil pressure of about 50 to 75 lb. per sq. in. These valves are opened successively one after another as their respective capacities are required under an increasing rate of discharge. Ordinarily only one, 6 ft. in diameter, is needed since this can take care of flows up to about 950 c.f.s. The other three, each 5 ft. in diameter, are available for the higher flows, although all four will not be used except when the available discharge is sufficient to tax the capacity of the tunnel in both directions. The maximum capacity after Quabbin Reservoir is completed and gates are installed at Shaft 1 will be about 1 000 m.g.d. eastward to Wachusett Reservoir and at the same time about 750 to 1 030 m.g.d. westward to Quabbin Reservoir depending on whether that reservoir is full or empty. To date, the maximum quantity discharged in 24 hours eastward, to Wachusett Reservoir only, is 682 m.g., the maximum discharge for a short period being at the rate of 1 200 m.g.d., which was sent through the tunnel on April 1, 1932, for test purposes.

A travelling bridge crane in the building has an electrically driven hoist which, when the valve pit is unwatered, can be used to lower men and materials through the shaft into the tunnel. This hoist can be rereaved in a ratio of one to four for use at a 7½-ton capacity for handling the valves and other heavy equipment. The travel of the bridge and of the hoist along the bridge is effected manually. The side-lights of the bronze main-entrance door can be removed to pass large equipment as in the case of the Wachusett Outlet building. Here also the roof-truss members were covered with a protective coating of Gunite.

The design of many portions of the entire project and particularly of these intake works involves a number of interesting features which cannot very well be discussed in a general description of this sort. Some of the more interesting details include a vacuum system piped throughout the building, connected to the Venturi meter which measures the 85-m.g.d. discharge down the river, at a point just downstream from the meter throat. This vacuum provides among other things a means for rapid priming of the siphons. The entire control mechanism for the four main valves in the bottom of the pit is on a single small panel which contains for each of the four units a small oil sleeve-valve and a simple device for preventing hunting or racing so that the valves adjust their openings easily and smoothly to the fluctuations of the flow in the river. Another small mechanical brain, in the form of a floating dash-pot, practically anticipates

the small rise and fall of the river and adjusts the speed of opening and closing the valves accordingly. By the use of specially designed super-sensitive float-switches operating on a differential head of a small fraction of an inch, the water level behind the diversion dam is kept practically constant during the entire range of operations from a diversion of nothing up to the entire capacity of the works. These switches assist the rapid priming of the siphons one after another as needed. The operation is entirely automatic with an operator on hand only during the ordinary working hours of the day.

FIG. 14.—CONSTRUCTION OF LOWER SWIFT RIVER INTAKE TO QUABBIN AQUEDUCT TUNNEL.
OCTOBER 17, 1932.

When flood flows have been available, the diversion has been continued without attendance nights, Sundays and holidays.

Design of the Swift River Intake. The intake works at Shaft 12 will consist of an upper intake to be constructed above the shaft itself drawing from the upper water levels at depths from 5 to 45 ft. below the reservoir flow-line and a lower intake now under construction extending 200 ft. from the west tunnel portal, and 520 ft. from the shaft, drawing normally at depths from 76 to 91 ft. or in an extreme drought to 102 ft. below the reservoir flow line. A coarse rack of reinforced concrete bars with 12-in. open spaces is built over the entrance to the lower intake. After the reservoir starts to fill, the only access to this rack will be by a diver, who may land and work on its flat top. The draft from both intakes will be controlled by shutters operated from a head house over Shaft 12. No gates, Venturi meters or other works will be installed. They would in any event be im-

practical on account of the two directional flows through the tunnel. The flow from the Ware River intake works westward into Quabbin Reservoir can be as much as 1 030 m.g.d., that eastward from Quabbin Reservoir to Wachusett Reservoir can be as much as 600 m.g.d. Emergency spillway wells from the basement of the building to the lower intake are provided, to by-pass any stop shutters that can be placed, so that in no event can a flood discharge from the Ware River exert a lifting pressure at the top of Shaft 12.

Design and Construction of Stream Control Tunnel at Main Dam Site. The only important item of construction that has been accomplished at the

FIG. 15.—COMPLETING PAVING AND RIPRAP AT ENTRANCE TO RIVER, DIVERSION TUNNEL, MAIN DAM SITE, QUABBIN RESERVOIR. JUNE 14, 1933.

main dam site is a tunnel through the rock under the west abutment of the dam, through which the river has been diverted from the site of future construction operations. The Swift River valley has not experienced any floods of great magnitude during the short period of record covering the last twenty years. However, the possibility of the occurrence of such floods is recognized, and the stream control tunnel, 30 ft. wide by 28 ft. high, 1 230 ft. long is required to make sure that the construction of the main dam can be completed expeditiously and all risk of damage from flood flows eliminated. The sound ledge on the side of the valley at this site is covered with huge irregular masses of glacial drift which made it necessary to excavate about 289 000 cu. yd. in the approach and discharge cuts. Furthermore, it was impracticable to secure a straight alignment for the tunnel. Under the

circumstances, the most efficient hydraulic properties for the discharge of the necessary 20 000 to 25 000 c.f.s. at high velocity were secured by building the tunnel in a straight line to a point near the downstream portal. Here a smooth gradual Venturi-like enlargement was obtained by lowering the floor of the tunnel about 6 ft. and raising the arch about 4½ ft., keeping the same dimensions of the arch for simplicity in form work. Under flood-flow conditions taxing the capacity of the tunnel, this enlargement will reduce the velocity without material loss of head. The alignment then contains a 90-deg. bend near the downstream end, but the losses in this bend are materially reduced on account of the decrease in velocity in the enlarged

FIG. 16.—DRILL CARRIAGES AT DOWNSTREAM PORTAL OF RIVER DIVERSION TUNNEL, MAIN DAM SITE, QUABBIN RESERVOIR. MAY 2, 1932.

section with depressed invert which is carried completely around the bend to the downstream portal.

After the main-dam embankment is built it will be necessary to plug this diversion tunnel with a solid block of concrete 40 ft. in length. In anticipation thereof 2 piers, at a point about 375 ft. from the upstream entrance, cut the waterway into 3 bays of 9-ft. span for the insertion of stop logs. To minimize the loss of head past these piers, the tunnel section upstream from the piers is 30 ft. high or 2 ft. more than downstream and the net water area past the piers is the same as that of the main tunnel downstream. Like the Ware River intake already described, many interesting features of the design cannot be discussed in a general description such as this. Provision had to be made not only for the future plugging of the conduit but for the needs of future operation and the release of stored water to meet the requirements of Massachusetts Legislation and of the War Depart-

ment previously referred to. These provisions include a vertical shaft, above the piers and the future plug, which will serve as a high-level intake, also a 48-in. pipe which had to be embedded at this time in the concrete side walls and invert of the tunnel extending from this future intake through to the downstream portal, where foundations were prepared for future control works. One of the novel features of the design is provision for maintaining the flow during future plugging operations by siphoning the river through a 48-in. pipe in order to by-pass the construction work required to complete the 40-ft. plug.

All materials excavated from the approach and discharge cuts were used to construct cofferdams, both upstream and downstream, forming the toes of the future dam. These were placed entirely west of the railroad which runs along the east bank of the river. They will later need to be extended across the entire valley, when the construction of the dam foundations is undertaken. The rock excavated from the tunnel was used to construct portions of the downstream toe and for riprap so far as needed, the balance being stored for future similar use. In excavating the tunnel, a top heading from about 2 ft. below the springing line of the arch to the roof was first excavated about 840 ft. from the downstream portal face followed by a pilot cut about 6 ft. square which was holed through to the upstream portal for ventilation only, the entire enlargement to full section being excavated from the downstream portal. The rock was sound throughout the entire length of the excavation, and the roof required no timber or steel support or Gunite protective treatment. The size of the tunnel allowed mucking operations with a steam shovel and heavy trucks. Special drill carriages were erected on truck bodies and thus moved rapidly and conveniently to and away from the face. The heading excavation was started January 4, 1932, holed through in 13 weeks, 955 ft., and the entire excavation of the full section was practically completed 11 weeks later, the shaft being worked from the tunnel up. The lining was cast with steel forms and required 12 weeks. A cut and cover section 275 ft. long extends to the upstream face of the embankment of the dam, the total length of the conduit being 1 230 ft. The tunnel was completed on February 8, 1933. Complete diversion of the river from its old bed through the tunnel was effected June 7 by small earth dams across the river bed near the end of the approach and outlet channels.

Design and Construction of Caisson Core Walls. The water-tight cutoff required along the center line of the dike from the original surface through the porous, glacial material to sound ledge is 130 ft. deep below the brook level. A similar cutoff at the main dam will need to extend to a depth of at least 110 ft. below the river bed. An exploratory caisson was first sunk, in the less difficult of the two situations, at the dam, a location being selected about 450 ft. from the river and at a point where ledge was encountered about 70 ft. below the river level and 102 ft. below the original surface, but about 45 ft. above the lowest point of the rock valley. A second exploratory

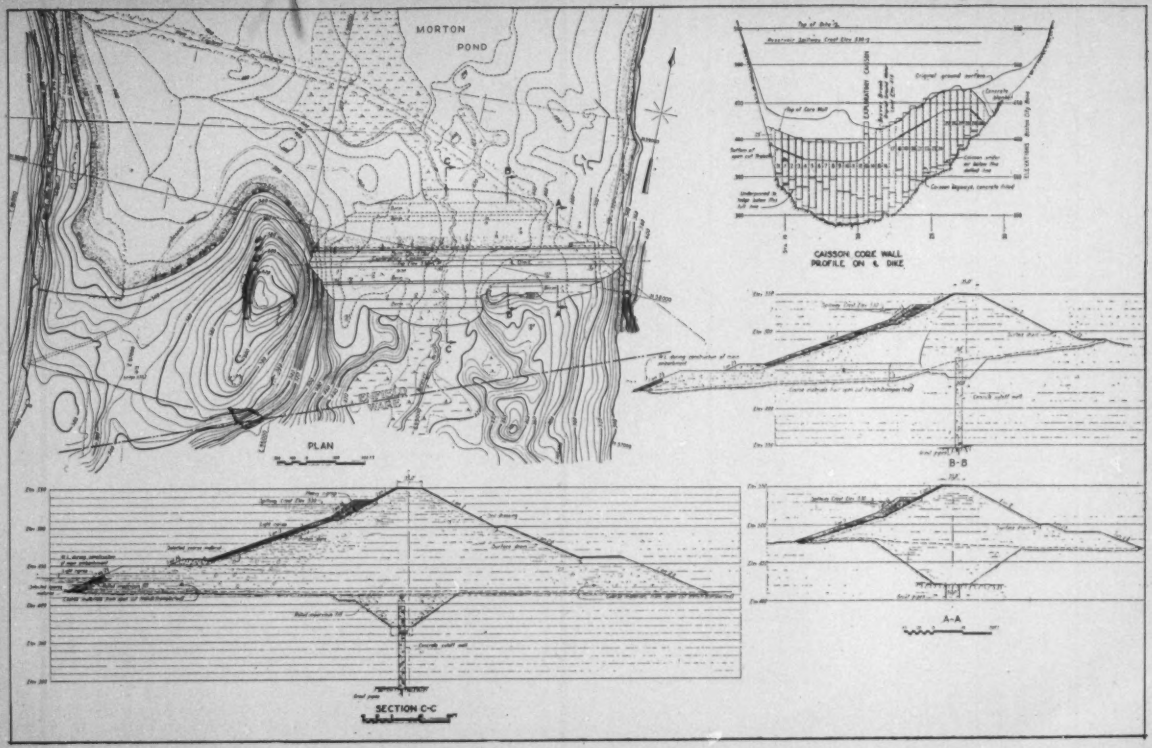

FIG. 17.— QUABBIN RESERVOIR — DIKE.

caisson was put down at the dike at practically the deepest point in the rock gorge 127 ft. below the original ground-water level. This required alternate pumping and excavation of the caisson and an auxiliary pump sump near the caisson to lower the ground-water level in successive stages so that the air pressure required to seal the caisson to ledge did not exceed 48 lb. per sq. in. The caisson itself was first sunk as deep as possible without resorting to excessive air pressures, about 30 lb. per sq. in. being used. By pumping from the working chamber, the ground-water level in the vicinity was then lowered, and a sheeted pump sump about 26 ft. square was ex-

FIG. 18.—CONSTRUCTION OF CAISSON CORE WALL TO SOUND LEDGE ON CENTER LINE OF QUABBIN RESERVOIR DIKE. JUNE 15, 1933.

cavated at a point about 100 ft. from the caisson to a depth of about 50 ft. below the original ground-water level. The pumps were then transferred to this sump and the lowering of the ground-water level continued during the further sinking of the caisson to ledge, requiring 48 lb. per sq. in. air pressure.

These first two exploratory caissons were 32 ft. long by 12 ft. wide, with two 7-ft. diameter wells for access to the working chamber and a central well to be used in pumping experiments. The results of sinking these caissons indicated that a 9-ft. width would give a working chamber with ample room, that access wells 4 ft. in diameter were large enough, and that a greater length of caisson could be handled without difficulty. Accordingly, the caissons designed for the dike core-wall were made 45 ft. long by 9 ft. wide with three 4-ft. diameter wells. In sinking these two exploratory caissons, various experiments were made to determine the feasibility

of combining pumping operations for lowering ground-water level with the sinking operations. It was found impracticable to sink caissons through this glacial material, containing many boulders of all sizes, without forcing the water level down so that considerable quantities of air escaped below the cutting edge of the shoe. It was also found that the action of the air escaping through the ground often had a tendency to separate the fine materials which were carried along in any underground water courses. These fines had a tendency to become tightly packed and clog any suction intakes above the cutting edge from which an attempt was made to pump the ground water. It was demonstrated that no pumping could be done from a caisson while it was being sunk under compressed air, but that all pumping for the purpose of lowering the ground-water level should be done from such an intake placed at an elevation lower than the cutting edge of any caisson being sunk in the immediate vicinity.

Accordingly, when the exploratory caisson at the dike was sealed to the ledge, four pump-suction intakes were permanently embedded in the seal consisting of pipes projecting into areas filled with selected coarse material. One 6-in. pipe intake was constructed under the cutting edge on each side and one 8-in. on each end. These were carried up into the bottom of one of the 7-ft. diameter wells. These pump-suction pipes were independent of each other so that upon completion each could be grouted to prevent any possible open waterway through the core wall. A pump was installed in the well and commencing the first week in September, 1932, water was pumped from the ground at the rate of 1 250 g.p.m., in addition to about 500 g.p.m. pumped from the open sump nearby, until this operation was taken over by the contractor for the caisson core wall on January 21, 1933. Meanwhile the brook was carried past the site in a small wooden flume in order to reduce seepage from the brook bed into the ground. When the contractor took over the work the ground-water level in the vicinity had been lowered 41 ft., and considerable data had been made available to prospective contractors as to the possibility of further unwatering by the installation of more pumping capacity.

A scheme for ground-water pumping operations was included in the contract plans by which the contractor could be paid for sinking at selected locations two of the caissons from the original surface, before excavating any open trench across the valley, in order to start working pumps in these caissons as soon as possible. As soon as the contract was let, the contractor took over the pumping operations and installed pumps of greater capacity in the deep dry well which had been prepared for him in the exploratory caisson. Instead of sinking the two caissons as proposed, the contractor elected to pump intermittently at shallower depths from a number of caissons at different stages of the sinking, starting first on the east side. After sinking one of these caissons by open digging until the ground-water level was reached and to such further depth as was convenient without using excessive air pressure, the sinking of this caisson was stopped for a time; a

specially designed steel diaphragm was installed in the bottom of one the circular access wells to the working chamber and a temporary pumping plant was installed in this well, pumping directly from the working chamber with an ideal large gravel floor surface for a suction screen. By this method the work progressed very satisfactorily. In addition to pumps installed above the temporary steel diaphragms in caissons not yet sunk to ledge, permanent suction intakes embedded in the concrete seal between the cutting

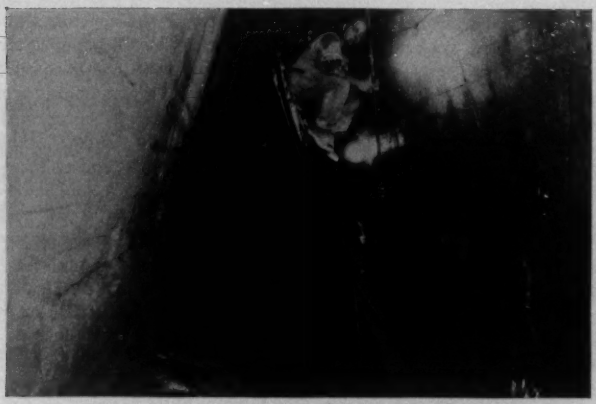

FIG. 19.—WORKING CHAMBER OF DIKE CORE WALL CAISSON AFTER COMPLETION OF SINKING AND UNDERPINNING FROM CUTTING EDGE TO LEDGE. AUGUST 4, 1933.

edge and sound ledge were installed in three completed caissons similar to those installed in the first exploratory caisson.

The total rate from all of these pumps has averaged about 3 200 g.p.m. for the last year. The ground-water level in the deep portion of the valley in the exploratory caisson has been lowered 96 ft. from the original ground-water level, a gain of about 55 ft. since the pumping was taken over by the contractor. The valley is about 1 500 ft. wide, and the natural slope of the original bed of the brook, which flows out of Morton Pond, about 1 200 ft. upstream from it, is about 1.5 ft. per thousand. The ground-water level has been lowered 56 ft., 220 ft. upstream, 37 ft., 800 ft. upstream, and 16 ft., 2 350 ft. upstream, and Morton Pond itself has been lowered by the underdraining operations about 9 ft. The ground-water level has been lowered 35 ft., 1 050 ft. downstream, and 14 ft., 2 050 ft. downstream. Our computations indicate that if the contractor had used higher air pressures on two caissons sunk at the start from the original surface the entire distance to

sound ledge in order to establish pumps at fixed strategic points, he might have accelerated the lowering of the ground-water level and effected a slight economy in his operations. Table 10 recording sinking progress shows that the air locks were put on quite a bit sooner than the pressures finally required to seal to ledge would indicate as necessary if the pumping had been started earlier.

It was estimated that an economical balance between the cost of the caissons sunk from the bottom of an open trench across the valley and the cost of excavating such a trench and refilling it finally with suitable impervious material would be obtained at a depth of about 45 ft. Accordingly, this depth to the bottom of the open-cut trench was specified in the contract as the pay line from which caissons should be sunk to sound ledge. The excavation of this trench was kept well in advance of the placing of the caisson shoes. Suitable previous materials from the excavations were used to construct embankments across the valley both upstream and downstream which will serve as toes for the future dike embankment. A layer of impervious material on the upstream toe converted this into a cofferdam which will cut off the entire flow of the brook when the construction of the future dike embankment is begun. At present an 18-in. pipe serves as a by-pass through the operations, but the only flow in the brook at present is that discharged from the pumps for ground-water lowering. A considerable quantity of material, too fine for use, was wasted in spoil areas downstream from the site.

The steel shoe or cutting edge for the first caisson was set March 22, 1933. This caisson was sunk from an elevation just above the original brook level to a depth of 48 ft., before ground water necessitated the installation of air locks. The contractor then continued with the caissons in the east half of the valley, sinking first every alternate caisson and later the intermediate ones. The concrete was poured in steel forms erected in 16-ft. lifts using two 8-ft. panels each, and one lift of about 210 cu. yd. was poured daily. Aggregates for the concrete were obtained from an excellent gravel deposit about one mile from the site. A central crushing, screening, washing and batching plant was installed just downstream from the site to which cement was hauled in bulk from the railroad siding at West Ware, three miles distant. Mixing was done enroute to the caissons in two 5-cu. yd. transit mixers. As soon as each caisson was sunk to the point where its cutting edge touched ledge, concrete underpinning walls were built extending until the entire area of the ledge was exposed and clean. A concrete seal was then poured to a height of 3.5 ft. above the cutting edge, leaving a sufficient area in the working chamber to operate drilling and grouting equipment by which seams in the ledge, pump suction openings, etc., were grouted. All the working chambers will ultimately be completely filled with concrete.

In the easterly half of the wall the underpinning and sealing to rock were usually started at the uphill end of the caisson and extended a little more than half the horizontal length or so as to furnish a firm support for the

weight of the caisson, and the seal was not completed until the next adjoining caisson on the lower or downward side had been similarly sealed on ledge and a firm foundation. Such completion of the seal was thus carried through to the adjoining caisson forming a continuous cut-off across the valley at the ledge. Above the ledge the cut-off was made continuous by filling the end joints between adjoining caissons. As previously stated, the caissons were 45 ft. long and 9 ft. wide. They were placed 46.5 ft. on centers, leaving an end clearance of 18 in. The ends were squared for a height of 7 ft. above the cutting edge. Above this an 18-in. recess was cast in each end. Hence the working space available for excavating and filling the end joints averaged 4½ ft. in width. This proved ample in spite of the fact that it was impossible to sink the caissons precisely on the vertical alignment and some of the end joints were narrower and some wider. The end joint-space would also be distorted in a case where one of two adjoining caissons leaned sideways from the true alignment. Such distortion in no case amounted to more than a few feet, the maximum about 5 or 6 ft. at the top and 3 or 4 ft. at the bottom, and no difficulty was experienced in excavating the joints and supporting the outside earth with timbering. When the joint excavation reached the ground-water level the excavated portion was filled with a concrete cap, keyed to the caissons as necessary to prevent blowing off, leaving a 3-ft. circular well at the top of which an air lock specially designed for the close quarters was installed to permit continuing the excavation to the ledge, or to concrete previously placed as described before.

The wall across the valley requires 30 of these caissons in addition to the first exploratory caisson. All of these have been sunk and sealed to the ledge, the first shoe having been set in March, five more in April, five in May, four in June, nine in July, two in August, three in September, and the last one in October, 1933. The only other work of caisson sinking remaining under this contract is two additional exploratory caissons at the main dam site which are now being started in the deepest part of the gorge where experimental work in the lowering of the ground-water level will be continued.

Each caisson in the dike core-wall was required to be left projecting at least 25 ft. above the bottom of the open-cut trench. Where the caissons were seated with the tops too low, they are being extended by pouring additional concrete to the required height. The end joints have been completed, April 21, 1934, to an aggregate height of about 80 per cent. of the required total. At the extreme ends of the row of caissons, a solid concrete wall of the same width is built in sheeted trench or open cut to ledge and is extended until it meets the sloping ledge hillside, and a concrete blanket covering the ledge and sealing all seams for a width of 20 ft. will later be extended up the hill along the center-line of the impervious core of the future dike embankment. Grouting seams in the ledge through drill holes generally about 8 ft. apart and 15 ft. deep is now under way.

Table 10 giving the depths to which the caissons were sunk shows the

level reached by each before and after ground water was encountered, necessitating the installation of air locks. This does not indicate the full extent to which the work benefited by the lowering of the ground-water level, since this lowering continued to reduce the necessary air pressures after the locks were installed. During the sinking operations, the average rate of sinking in the open was 8.63 ft. in 24 hours and under air 2.51 ft. up to November 30, 1933. Although the maximum depth of sinking below the original ground-water level was 130 ft., the pressure (except for the exploratory caisson which reached 48 lb. per sq. in., as previously described) has rarely exceeded the 18 lb. per sq. in. under which men are permitted to work 8 hours a day, as many as in the open air. The maximum pressure reached on the caisson core wall contract was 25 lb. per sq. in.

In no case was it necessary to add pig iron or other excess weight to sink the caissons. However, whenever boulders too large to be handled in the bucket hoist or through the muck lock were encountered they were broken up with dynamite, and this blasting shook up the ground and helped settle the caisson. In some cases also, dynamite was exploded in the ground outside the cutting edge for the sole purpose of shaking up the ground and assisting in the sinking operation. In this connection it should be noted that the use of too much dynamite, particularly before the cutting edge approaches the required depth, may result in settling and packing the ground against the caisson so firmly as to be of doubtful benefit in completing the sinking. Several caissons were stopped a few feet before the cutting edge struck ledge because the Contractor chose to excavate the remaining distance behind underpinning walls rather than to carry the sinking further. The weight of the 25-ft. height of caisson required to be left projecting above the bottom of the open trench was in all cases available, even in the early stages of sinking, to such extent as the contractor chose to add this weight.

The following tabulation shows the distribution of the total Electrical Energy used on the Dike Caisson Core Wall including sinking of Exploratory Caisson and all pumping:

		k.w.h.			k.w.h.
1932	Feb. Exploratory Caisson	6 200	1933	Mar. Caisson Core Wall	68 130
	Mar.	53 700		Apr.	98 100
	Apr.	47 500		May	146 800
	May	67 900		June	158 800
	June	75 500		July	167 600
	July	143 700		Aug.	170 200
	Aug.	27 600		Sept.	216 900
	Sept. Interim Pumping	29 300		Oct.	214 300
	Oct.	43 000		Nov.	246 200
	Nov.	41 300		Dec.	254 000
	Dec.	42 300			
			1934	Jan.	262 400
1933	Jan. Caisson Core Wall	38 200		Feb.	247 400
	Feb.	42 100		Mar.	258 600

TABLE 10.— QUABBIN RESERVOIR DIKE, PROGRESS IN SINKING CAISSONS.

Caisson Number.	Date when shoe was set. 1933.	Feet Depth Sunk Below Bottom of Open Trench. (Finished caissons are left 25 ft. above bottom of open trench.)					Maximum depth in feet reached below original ground-water.	Maximum lbs. per sq. in. air pressure at lowest point reached.	Date of sealing to ledge. 1934.	Average feet per week, including total time of construction sinking, pumping and sealing to ledge.	Underpinning Extended to Next Caisson.	
		Without Air Locks.		Additional Under Air.		Total to lowest point reached.					1933.	Air Pressure lb. per sq. in.
		To original ground-water level El. 416.	Below original ground-water level.	Until sinking stopped.	Under-pinning below shoe.							
West End 31	Oct. 11	...	34	1	16	51	79	11	Jan. 26	3.3	.....	...
1	Sept. 11	...	43	3	9	55	87	11	Jan. 15	3.0	.....	...
2	Sept. 7	...	35	12	19	66	101	14	Jan. 4	3.8	.....	...
3	Sept. 5	...	40	17	12	69	108	18	Jan. 12	3.7	.....	...
4	Aug. 30	...	36	25	7	68	111	18	Jan. 21	3.3	.....	...
5	July 28	...	23	39	12	74	119	12	Mar. 18	2.2	.....	...
6	July 26	...	28	42	6	76	120	18	Mar. 22	2.2	.....	...
7	July 21	...	36	34	9	79	125	21	Feb. 15	2.7	.....	...
8	July 19	...	24	47	13	84	130	20	Mar. 24	2.4	.....	...
9	July 18	...	38	34	11	83	130	19	Mar. 15	2.4	.....	...
10	July 15	...	38	34	9	81	128	25	Feb. 23	2.6	.....	...

	11	July 12	...	43	29	12	84	130	21	Feb. 4	2.8	.....	...
	12	Aug. 29	...	53	24	10	87	128	24	Jan. 11	4.5	.....	...
	13	Exploratory	Caisson	..	..	..	..	(127)	(48)	1933	..	.....	...
	14	May 22	...	60	21	12	93	125	17	Sept. 21	5.3	.....	...
	15	June 10	...	69	16	10	95	121	17	Oct. 14	5.2	.....	...
	16	May 19	...	60	31	5	96	116	22	July 22	10.3	.....	...
	17	May 31	...	71	19	11	101	116	16	Nov. 8	4.4	.....	...
	18	Mar. 22	...	48	42	15	105	115	11	Nov. 27	2.9	.....	...
	19	June 23	...	81	8	13	102	105	4	Dec. 6	4.3	.....	...
	20	Apr. 26	2	69	14	10	95	93	11	June 3	17.0	Dec. 9	7
	21	June 5	7	72	0	12	91	84	10	July 27	12.1	.....	...
	22	Apr. 26	13	61	4	8	86	73	11	June 3	15.4	July 29	11
	23	June 12	19	53	6	7	85	66	6	Aug. 5	10.8	.....	...
	24	Apr. 19	24	52	4	7	87	63	13	June 10	11.5	Aug. 7	7
	25	May 27	25	48	3	8	84	59	8	July 8	13.7	.....	...
	26	Apr. 27	25	38	0	19	82	57	13	June 15	11.5	Aug. 11	7
	27	May 22	25	25	2	13	65	40	7	Aug. 16	5.2	.....	...
	28	Apr. 27	26	11	4	17	58	32	11	May 26	13.6	Aug. 24	5
East	29	July 14	26	3	2	10	41	15	6	Sept. 2	5.6	.....	...
End	30	July 10	12	0	(No Air)	14	26	10	0	Oct. 13	1.9	.....	...

In a contract of this character the method of payment is very important. A price of ten cents per million foot-gallons for the pumpage was fixed before bids were opened, and the prospective contractors bid unit prices for the excavation to fixed levels, the measurement being regardless of whether the pumpage had made it possible to excavate in the open or not. Two unit prices were bid for concrete, one for that placed in the open and one for that placed under air. It is believed that by this arrangement the preliminary experimental work was capitalized to the fullest extent and the contractor was encouraged to perform the work in the most economical manner.

The principal items of the contractor's plant are:

3 Low-pressure air compressors with a total capacity of 2 550 c.f.m.
1 High-pressure air compressor with a total capacity of 500 c.f.m.
32 Feet height of steel caisson forms.
4 Muck locks, 2 man locks.
10 Ground-water lowering pumps, each 800 g.p.m., at 90-ft. head.

Upon the completion of the caisson wall the wells in the caissons and the open trench in and around the projecting tops of the caissons will be backfilled with impervious soil to an elevation somewhat higher than the brook bed. The entire area between the toes of the future dike will be left in readiness for a contract for the construction of the dike embankment.

PART IV

Costs and Miscellaneous Data.

In the following are presented miscellaneous data on costs, and other data on the Metropolitan Water District not readily found elsewhere.

Table 11.—Contracts for the New Supply.

No.	Description.	Date.	Contractor.	Cost
1	Borings on Tunnel Line	Nov. 15, 1926	Pennsylvania Drilling Co.	$18 641.69
2	Borings at Dam and Dike	Nov. 12, 1926	Sprague & Henwood, Inc.	26 885.83
3	Aerial Photographs	Nov. 22, 1926	Fairchild Aerial Surveys, Inc.	5 997.50
4	Shaft 5 and 1 367 ft. Tunnel Excavation	Mar. 30, 1927	James J. Coughlan & Sons, Inc.	290 581.99
5	Cordaville Pipe Line	Apr. 22, 1927	Lock Joint Pipe Co.	83 762.86
6	Cordaville Pumping Station	May 12, 1927	F. A. Mazzur Co., Inc.	17 385.04
7	Hopkinton Pipe Line	May 7, 1927	Cenedella & Co.	134 365.72
8	Shafts 6 and 7 and 3 062 ft. Tunnel Excavation	Jun. 2, 1927	James J. Coughlan & Sons, Inc.	456 784.26
9	Aerial Photographs	Mar. 30, 1927	Fairchild Aerial Surveys, Inc.	2 560.00
10	Transmission Line for Construction of Wachusett-Coldbrook Tunnel	Apr. 27, 1927	New England Power Co.	Construction 86 439.58 Maintenance 33 107.81
11	Ashland Pipe Line	Jun. 8, 1927	Antonio Mogavero	130 145.35
12	Shafts 2, 3 and 4 and 3 810 ft. Tunnel Excavation	Jun. 1, 1927	Dravo Contracting Co.	632 727.44
13	Whitehall Pipe Line	Jul. 20, 1927	Portland Contracting Co., Inc.	60 603.17
14	East Portion Wachusett-Coldbrook Tunnel	Apr. 30, 1928	West Construction Co.	3 515 787.08
16	Whitehall Open Channel	Sep. 28, 1927	Sanders Eng. Co.	32 695.16
17	West Portion Wachusett-Coldbrook Tunnel	Apr. 30, 1928	West Construction Co.	4 127 094.19
18	Iron Castings for Ware Intake Works	Jan. 14, 1929	Barbour Stockwell Co.	19 431.07
19	Dam and Substructure, Ware Intake Building	Jul. 23, 1929	J. W. Bishop Co.	285 580.49
20	Coldbrook-Swift Tunnel	Apr. 23, 1931	Wenzel & Henock Construction Co.	4 978 031.80*
21	5.2 miles Ware-Belchertown Highway	Jul. 20, 1931	Lane Construction In addition to Dept. Public Works' share	129 980.25 95 726.94
22	Unwatering Pump at Shaft 1	Nov. 14, 1929	F. A. Mazzur Co., Inc.	13 371.76
23	Borings at Dam and Dike	Nov. 13, 1929	Sprague & Henwood, Inc.	47 500.00†
24	Superstructure, Ware Intake Building	Jul. 23, 1930	Munroe & Westcott, Inc.	70 114.04
25	Superstructure, Wachusett Outlet Building	Jul. 23, 1930	Munroe & Westcott, Inc.	62 329.23
26	Wiring and Electrical Equipment, Wachusett Outlet Building	Sep. 24, 1930	Anderson-Coffey Co., Inc.	10 685.21
27	Covering Swamps along Ware River	Sep. 23, 1930	Raymond A. Bergesson	4 395.51
28	Grading Grounds, Ware Intake Building	Aug. 17, 1931	Anthony Ross & Son	8 164.14

* Basis of award, contract not yet completed.
† Estimated, contract not yet completed.

No.	Description.	Date.	Contractor.	Cost.
29	Grading Grounds, Wachusett Outlet Building	Aug. 17, 1931	Anthony Ross & Son	11 852.68
30	Stream Control Works, Main Dam	Aug. 1, 1931	Northern States Contracting Co.	544 610.60
31	Head House, Shaft 4	Aug. 26, 1931	E. D. Ward Co.	5 689.02
32	Exploratory Caissons, Main Dam and Dike	Sep. 16, 1931	Dravo Contracting Co.	162 838.33
34	Driveways, Quabbin Park Cemetery	Oct. 29, 1931	Lane Construction Corp.	7 842.94
35	Electric Power for Pumping at Dam Site	Jun. 2, 1932	New England Power Co.	Scheduled Rates
36	Caisson Core Wall, Dike	Dec. 28, 1932	West Construction Co.	999 265.00*
37	Electric Power for Pumping at Dike Site	Sep. 21, 1932	New England Power Co.	Scheduled Rates
39	Rutland-Holden Sewer	Jul. 11, 1933	James J. Coughlan & Sons, Inc.	184 327.50*
40	Transmission Line Relocation	Jun. 20, 1933	New England Power Co.	Construction 104 790.10 Damages 50 000.00
41	9.1 miles Belchertown-Pelham Highway (including Department Public Works' share)	Jul. 20, 1933	Carlo Bianchi Co., Inc.	222 335.50*
42	Sewage Disposal Agreement with City of Worcester	May 1, 1933		325 000.00
43	7.6 miles Extension of Belchertown-Pelham Highway to New Salem Center (including Department of Public Works' share)	Aug. 25, 1933	V. Barletta Co.	300 397.50*
44	4.1 miles Extension of Belchertown-Pelham Highway North of New Salem (including Department of Public Works' share)	Nov. 15, 1933	John Iafolla Construction Co.	179 480.50*
45	Exploration of Dam and Dike, Borrow Areas	Feb. 7, 1934 B.	Perini & Sons, Inc.	6 200.00*

In general it is believed that payments to contractors on the work have included a fair profit to them.

* Basis of award, contract not yet completed.

TABLE 12.— SELECTED UNIT BID PRICES.

Description.	Unit.	Contract Price in Dollars, followed by Contract Number in Parentheses.
Borings		
Wash borings, 2½-in. extra heavy pipe	lin. ft.	2.50(1) 4.35(2)
Core borings, generally 20 ft. in sound bedrock, 1-in. diamond drill core:		
Surface to sound bedrock	lin. ft.	3.50(1) 4.35(2)
In sound bedrock, including boulders over 2 ft.	lin. ft.	4.50(1) 4.35(2)
Casing ordered left in place	lin. ft.	0.80(1, 2)
Hard Rock Tunnel		
Earth excavation for tunnel shafts	cu. yd.	6.50(4, 8) 10.00(12) 8.00(14) 7.00(20)
Rock excavation in shafts, and tunnel within 50 ft. of shafts	cu. yd.	14.90(4) 13.65, 14.00(8) 13.00, 13.50(12) 12.00(14, 17) 14.00(20)
Excavation in aqueduct tunnel except within 50 ft. of shaft	cu. yd.	10.00, 13.00, 13.50(12) 10.75(14) 11.00(17) 8.80(20)
Rock excavation in stream-control tunnel	cu. yd.	4.00(30)
Allowance for drainage in shafts and aqueduct tunnel	lin. ft.	22.20(4) 9.75(8) 15.00(12) 2.00(14, 17) 3.00(20)
Forms for shaft lining	lin. ft.	15.75(4, 8, 14, 17) 20.00, 22.00, 30.00(12) 15.00(20)
Concrete lining of construction shafts, except cement and grouting	cu. yd.	11.55(4, 8) 15.00, 16.00, 17.20(12) 8.00(20)
Concrete in special construction at shafts, except cement and reinforcement	cu. yd.	10.00(14, 17) 11.00(20) 6.80(30)
Forms for aqueduct tunnel lining	lin. ft.	2.60(14) 3.50(17) 1.11(20)
Concrete lining of aqueduct tunnel, except forms, cement and grouting	cu. yd.	7.50(14) 8.00(17) 3.00(20)
Concrete lining of stream-control tunnel and cut and cover section, except cement and grouting	cu. yd.	5.45(30)
Portland cement	bbl.	2.60(4, 17) 2.70(8) 3.20(12) 2.65(14) 2.15(20) 2.50(30) 2.70(36)
Permanent timbering in tunnel	M ft. B.M.	35.00(14) 25.00(17) 45.00(20) 100.00(30)
Temporary timbering in tunnel	M ft. B.M.	100.00(14, 17) 45.00(20)
Structural-steel roof-support in tunnel	lb.	.06(30)
Gunite protective coating of aqueduct tunnel rock roof	lin. ft.	2.50(20)
Gunite protective coating of stream-control tunnel rock roof	lin. ft.	12.00(30)

TABLE 12.—SELECTED UNIT BID PRICES—(Continued).

Description.	Unit.	Contract Price in Dollars, followed by Contract Number in Parentheses.
Open Cut Excavation and Embankment (Earth includes boulders up to ½ cu. yd.)		
Miscellaneous earth excavation	cu. yd.	1.00(14) 2.00(17) 1.50(20)
Earth excavation in large open cuts	cu. yd.	.21(30) .19, .20(36)
Removal of soil	cu. yd.	1.00(14, 20) .53(30) .25(34)
Stripping top 2 ft.	cu. yd.	.20(36)
Miscellaneous rock excavation	cu. yd.	4.00(14, 17) 6.00(19, 20)
Rock excavation from large portal cuts	cu. yd.	2.00(30)
Rock excavation, unrestricted blasting not permitted	cu. yd.	8.00(36)
Consolidated refilling and embanking	cu. yd.	1.00(14, 17) .50(20) .13(30) .12(36)
Unconsolidated refilling and embanking	cu. yd.	.10(30, 36)
Surface dressing of top soil	cu. yd.	1.00(14) 1.50(20)
Impervious filling in deep caisson wells	cu. yd.	1.00(36)
Rolled impervious embanking	cu. yd.	.40(30) .36(36)
Loam dressing	cu. yd.	.55(28, 29) .60(34) 1.00(39)
Seeding and grassing	acre	500.00(14, 20, 28, 29) 250.00(34) 200.00(39)
Masonry, Stone, Cement		
Miscellaneous concrete except reinforcement and cement	cu. yd.	15.00(14) 10.00(17) 12.00(19) 7.20(30) 7.50(39)
Steel for reinforcing concrete	lb.	.08(14) .05(17, 20) .045(19) .04(30) .03(39)
Portland cement	bbl.	2.65(14) 2.60(17) 2.15(20) 2.50(30) 2.70(36) 2.50(39)
Ashlar, except copings, dimension stone, and cement for joints	cu. yd.	35.00(14) 65.00(19)
Ashlar copings, except cement for joints	cu. yd.	50.00(14, 17)
Dimension stone	cu. yd.	165.00 (19)
Miscellaneous stone masonry, except cement for joints	cu. yd.	60.00(20)
Stone paving	cu. yd.	5.00(14, 17) 4.00(19) 3.00(20) 6.00(30) 3.00(39)
Concrete channel paving, except cement	cu. yd.	4.55(30)
Rip rap	cu. yd.	2.00(19) 1.00(20) 2.25(30) 2.50 (36) 2.00(39)

Sinking Caissons (Contractor is also paid 10 cents per million feet-gallons for pumping to lower the ground water. Earth includes boulders up to ½ cu. yd.)		
Earth excavation in caissons from variable elevation of bottom of open trench to 106 ft. below original ground-water level	cu. yd.	5.85(36)
Rock excavation in caissons to 106-ft. level	cu. yd.	9.00(36)
Earth excavation in caissons below 106-ft. level	cu. yd.	13.00(36)
Rock excavation in caissons below 106-ft. level	cu. yd.	14.00(36)
Preparing ledge surface for seal to 106-ft. level	sq. yd.	2.00(36)
Preparing ledge surface for seal below 106-ft. level	sq. yd.	5.00(36)
Concrete including cutting edge, air lock wells, and reinforcement, except cement	cu. yd.	5.60(36)
Grouting rock seams, except drilling and cement	cu. yd.	7.00(36)
Portland cement	bbl.	2.70(36)
Pipe Lines		
Excavating and refilling pipe trench in earth	cu. yd.	1.25(5) 1.49(11)
Excavating and refilling pipe trench including ledge and boulders	cu. yd.	2.85, 4.90(7) 1.22, 2.40(13)
Excavating and refilling pipe trench in earth for 10-in. to 16-in. pipe	lin. ft.	.68(39)
Furnishing 30-in. reinforced concrete "Cylinder" pipe, including specials	lin. ft.	6.60(5)
Laying 30-in. reinforced concrete "Cylinder" pipe, including specials	lin. ft.	.45(5)
Gunite lined 48-in. steel pipe, embedded in tunnel lining	lin. ft.	13.00(30)
Furnishing 30-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	6.00(7)

TABLE 12.—SELECTED UNIT BID PRICES—(Concluded).

Description.	Unit.	Contract Price in Dollars, followed by Contract Number in Parentheses.
Furnishing 24-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	5.30(11)
Furnishing 20-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	4.25(13)
Laying 30-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	3.00(7)
Laying 24-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	.65(11)
Laying 20-in. C. I. pipe, A. W. W. A., Class A, including specials	lin. ft.	.92(13)
Furnishing and Laying 16-in. C. I. pipe, Sandspun	lin. ft.	2.30(39)
Furnishing and Laying 12-in. C. I. pipe, Sandspun	lin. ft.	1.60(39)
Furnishing and Laying 10-in. C. I. pipe, Sandspun	lin. ft.	1.25(39)
State Highway Construction		
Roadway earth excavation	cu. yd.	.45(21) .19(41) .20(43) .27(44)
Ledge excavation	cu. yd.	.45(21) 1.00(41, 44) 1.15(43)
Ordinary borrow	cu. yd.	.15(21) .19(41) .10(43) .20(44)
Broken stone	ton	2.50(21) 1.40(41) 1.55(43) 1.80(44)
Bituminous material	gal.	.08(21) .105(41, 43) .09(44)
Gravel borrow	cu. yd.	.40(21) .24(41) .30(43, 44)
Sand borrow	cu. yd.	.50(21) .60(41) 1.00(43) .30(44)
Loam borrow	cu. yd.	.80(41, 44) .60(43)
Fine grading, rolling and finishing	sq. yd.	.03(21) .01(41, 43) .005(44)
Cobble guard rail	lin. ft.	.40(21) .35(41) .36(43) .33(44)
Miscellaneous		
Cast-iron shaft lining plates	ton	63.70(18)

It is difficult to estimate from bid unit prices alone, the total amount actually paid for certain units of the work. There follows a summary of selected average prices paid:

Aqueduct Tunnel. The total amount paid for sinking and lining the 14-ft. diameter *construction shafts* ready for tunnel excavation was $192 per foot for the aggregate depth of 2 608 ft. sunk under the preliminary shaft contracts, Nos. 4, 8 and 12, and $150 per foot for the aggregate depth of 883 ft. sunk under the tunnel contract, No. 20. The amount paid for excavating tunnel under the shaft contracts, without shaft head frames and regular tunnel excavating equipment, averaged $107 per linear foot.

Under Contract 20 the price paid for permanent timbering per foot of timbered tunnel was $9.30 for the timber in place and at least $6.10 for the additional excavation necessitated by the timbering. The amounts paid for *excavating and lining the tunnel* under the tunnel contracts were as follows:

	Per Linear Foot	
	Contracts 14 and 17	Contract 20
Net length between the shaft bottoms — ft.	66 512	54 431
Excavation	$76.80	$63.30
Permanent and temporary timbering and Gunite treatment	.10	.65
Distributed cost of the transmission line under Contract 10	1.60	..
Total length lined, or to be lined — ft.	74 870	54 862
Concrete lining, except cement	$19.80	$7.60
Cement	7.95	6.50
Miscellaneous items of sanitary, medical, hospital and other services, cleaning up, etc.	.45	.45
Total	$106.70	$78.50

Main Dam Stream Control Tunnel, Contract No. 30. Earth excavated from the approach cuts was used to construct the cofferdam embankments or toes of the future main dam embankment, and the amounts paid for such work are stated in the bid prices. As for the tunnel itself, *rock* excavation included the two portal cuts, the shaft, and the tunnel bore 955 ft. long. The upstream portal was carried 100 ft. farther into the ledge than would have been necessary if it had not been for the softness of the rock near the ledge surface at this point. For this excavation $198 390 were paid which is $208 per linear foot of tunnel bore or $3.26 per cubic yard of ledge in the bore and the portal cuts. In addition $223 735 were paid for concrete in the *lining* of the tunnel and shaft, the 275-ft. cut-and-cover extension of the tunnel bore, and the portal wing walls, including embedded pipes, stop-log grooves, structural steel in the closure piers, etc. This is $182 per linear foot of the completed structure between the entrance and exit faces, or about $10.80 per cubic yard of all concrete placed downstream from the approach channel. In addition $8 870 were paid for miscellaneous items of sanitary

and other services, river control contingencies, cleaning up, etc., so that the total construction cost, excluding the net cost of the earth-handling items, was $430 993 or $350 per linear foot of the total length of 1 230 ft.

Sinking Caissons, Contract No. 36. The total amount estimated for payment to date for sinking caissons, underpinning to ledge and sealing with concrete sufficiently to allow the air pressure to be taken off, in preparation for but not including drilling and grouting, also for pumping, concrete required for caisson projections 25 ft. above the bottom of the open-cut trench and for core wall completed at east end, and excavating and filling the end joints, was $647 812. This includes all the caisson sinking required in the dike core-wall and about 80 per cent. of the total excavation and filling of end joints required. The total amount of work for which this total sum was paid is divided as follows:

	Vertical Core Wall Area, Square Feet	Volume in Specified 9-ft. Width, Cubic Yards
Between stated levels:		
Above the original ground-water level	8 600	2 865
Between the original ground-water level and the specified payment line 106 ft. down	82 850	27 620
Below the 106-ft. depth	9 200	3 065
Total	100 650	33 550
With and without air pressure:		
Open caisson excavation (above the level to which the lowered ground water was encountered and the use of air locks necessitated.)	67 250	22 420
Pressure caisson excavation:		
Above the 106-ft. level	24 200	8 065
Below the 106-ft. level	9 200	3 065
Total	100 650	33 550
Total cubic yards of concrete in caisson projections above bottom of open-cut trench	..	11 550

Miscellaneous Heavy Equipment. The following items of equipment were purchased directly by the Commission for installation by its contractors:

20-ton hand operated travelling crane with one 5-ton electric hoist, installed in Wachusett outlet building under Contract No. 25. Span 32 ft. 9⅞ in. center to center crane rails, 1930. $3 534 delivered.

7½-ton electric travelling crane. Installed in Ware River intake building under Contract No. 24. Span 58 ft. 9⅞ in. center to center crane rails, 1930. $3 850 delivered.

4 Dow-disc butterfly valves, one 72-in. and three 60-in. including operating and stand-by oil pumping units, control equipment and all auxilliary apparatus, 1930. $28 267

220-ft. deep-well motor-driven pump, 4¾ in. by 24 in., 110 g.p.m., except the well casing in the tunnel dry-well at Shaft 1, 1929. $1 700 delivered.

The 900-h.p. motor-driven tunnel-unwatering pump and auxilliary equipment at Shaft 1 was not purchased separately by the Commission but

was furnished and installed under Contract No. 22. Also the 250-h.p. motor-driven pump in the Cordaville Pumping Station was furnished and installed under Contract No. 6.

TABLE 13.— EXPENDITURES FOR WATER SUPPLY WORKS.

Spent for the Wachusett-Sudbury-Cochituate Supply, and the Distribution Works.		
40-year sinking-fund bonds, issued 1895 to 1910		$41 398 000
30 to 40-year serial bonds, issued 1911 to 1925		4 287 000
Expenditure of current funds 1909, 1925 to 1934, including $300 000 requested appropriation for 1934		4 245 000
		$49 930 000
Paid for, to date.		
Adjustment of sinking fund, January 1, 1903	$2 256 804	
Assessments paid into sinking fund, 1903 to 1934	9 634 081	
Fees, net earnings of Wachusett and Sudbury plants, etc., paid into sinking fund 1903 to 1933 (to Nov. 30)	2 669 459	
Accumulated interest on sinking fund investments, 1903 to 1934 (including miscellaneous earnings since Nov. 30, 1933)	18 633 242	
Total sinking fund, April 1, 1934	$33 193 586	
Assessments to pay serial-bond annuities, 1911 to 1934	1 656 000	
Entrance fees, applied to construction, 1909, 1925	490 000	
Assessments to "pay-as-you-go," 1926 to 1934	3 755 000	
		39 094 586
Net water debt, April 1, 1934, account of Old Supply, after deducting $114 000, 1934 assessment to pay serial bond annuities due Jan. 1, 1935		$10 835 414
Spent for the New Ware-Swift Supply. (This includes both the cost of the Southern Sudbury Emergency Supply and a credit for the transfer to Worcester of rights in the Wachusett watershed.)		
20-year serial bonds, issued 1927		$1 000 000
30-year serial bonds, issued 1928 to 1934		27 500 000
Direct expenditure, 1927		1 000 000
		$29 500 000
Paid for, to date.		
Assessments to pay serial-bond annuities, 1928 to 1934	$3 451 000	
Worcester's fee, applied to construction, 1927	1 000 000	
		4 451 000
Net water debt, April 1, 1934, account of New Supply, after deducting $339 000, 1934 assessment to pay serial-bond annuities due July 1		$25 049 000

Approximate Distribution of Expenditures for Water Supply Works	
Wachusett-Sudbury-Cochituate Supply and Distribution Works:	
Wachusett Reservoir System	$12 786 000
Wachusett Aqueduct	1 798 000
Sudbury Reservoir System	7 655 000
Sudbury Aqueduct	3 705 000
Weston Aqueduct and Reservoir	7 031 000
Cochituate System and Aqueduct	1 869 000
Chestnut Hill and Spot Pond reservoirs, pumping stations, distribution system, and miscellaneous	11 906 230
Share in Metropolitan District Commission Building	250 000
Mystic River System and other old works	1 500 000
Total capital expenditures, Nov. 30, 1933	$48 500 230
Unexpended balance, and miscellaneous adjustments in the capital account	1 129 770
Capital account Nov. 30, 1933, not including $300 000 requested appropriation for 1934	$49 630 000
New Ware-Swift Supply (Including Southern Sudbury Emergency Supply).	
Construction of Quabbin Reservoir:	
Contracts for main dam	$599 770
Contracts for dike	804 980
Contracts for highway relocation	296 030
Contracts for electric transmission-line relocation	104 790
Contracts and force account for Quabbin Park Cemetery and cemetery removals	43 090
Miscellaneous equipment and construction	11 480
Construction of Quabbin Aqueduct:	
Contracts for Wachusett-Coldbrook tunnel section including lining of Shafts 1 and 8	9 161 950
Contracts for Ware River intake works	363 860
Contracts for Wachusett outlet works	98 240
Contract for head house at Shaft 4	5 690
Miscellaneous equipment and construction	90 630
Contract for Coldbrook-Swift tunnel section including Swift intake substructures	3 910 080
Watershed protection, real estate and damages for Ware-Swift supply	7 803 610
Engineering, overhead and administration for Ware-Swift supply	2 815 940
Southern Sudbury Emergency Supply	638 530
Joint Board investigation of watersheds	2 000
Construction of Rutland-Holden sewer	96 180
Total capital expenditures to Mar. 31, 1934 (Less $361 720 miscellaneous receipts credited to construction fund, and not including $450 620, estimated for payment on contracts but not yet paid.)	$26 846 850
Unexpended balance of bond issues	2 653 150
	$29 500 000

TABLE 14. — METROPOLITAN WATER DISTRICT ANNUAL ASSESSMENTS.

(Proportioned to the member municipalities, one-third on valuation and two-thirds on consumption.)

Date of Assessment, April 1.	Assessment.					Method of Determining Approximate Share of Each Municipality in Dollars.
	For Maintenance.	For bonds, all interest, and pay-as-you-go expenditures.		Credit by fees received.	Total.	P = Population in previous year in thousands. V = Valuation in previous year in million dollars. C = Consumption in previous year in m.g.d.
		Construction of old supply and distribution works.	Construction of new Ware-Swift supply.			
1905	$328 624	$1 848 962	.....	.....	$2 177 586	2 178 × (0.2176V + 0.4699P)*
1910	380 242	1 917 546	.....	.....	2 297 788	2 298 × (0.1958V + 5.596C)
1915	417 623	1 746 707	.....	.....	2 164 330	2 164 × (0.1673V + 6.227C)
1920	746 853	1 771 431	.....	.....	2 518 284	2 518 × (0.1656V + 5.528C)
1925	741 180	2 186 236	.....	$400 000	2 527 416	2 609 × (0.1273V + 5.209C)**
1926	769 799	2 143 553	.....	.....	2 913 352	2 996 × (0.1217V + 5.035C)
1927	847 554	2 244 488	$1 000 000	1 000 000	3 092 042	3 192 × (0.1171V + 4.975C)
1928	886 454	2 480 132	82 570	.....	3 449 156	3 551 × (0.1133V + 4.876C)
1929	825 574	2 318 569	432 697	.....	3 576 840	3 692 × (0.1108V + 4.759C)
1930	892 908	2 246 158	821 193	.....	3 960 259	4 082 × (0.1101V + 4.703C)
1931	966 102	2 292 221	953 919	.....	4 212 242	4 343 × (0.1080V + 4.724C)
1932	926 934	2 273 459	1 247 973	.....	4 448 366	4 598 × (0.1083V + 4.776C)
1933	857 624	2 079 536	1 504 215	.....	4 441 375	4 603 × (0.1109V + 5.025C)
1934	.....	2 089 377	1 763 390	.....	.....	.....

* Boston's share in 1905 = approximately 2 178 × (0.6527V).

** Brookline's share in 1925 (before deducting credit for water supplied from local sources) = approximately 2 609 × (0.1232V + 5.043C).

TABLE 15.— GROWTH OF THE TWENTY CITIES AND TOWNS *NOW* IN THE METROPOLITAN WATER DISTRICT.

(With the exception of Swampscott, admitted in 1909, and Brookline, admitted in 1925, all were in the original District or admitted prior to 1903.)

Year.	Population.	Gallons per capita.	Consumption — m.g.d.			
			"Pump basis" to 1903, "Meter Basis" 1904 to date.	Members taking their entire supply from the District.	Total water-supply demand on District sources, including sales outside the District, etc.	
					Av. Annual.	Maximum Month.
1850	226 540	....	.....	.....	.....	...
1860	321 189	....	.....	.....	.....	...
1870	389 625	....	.....	.....	.....	...
1880	508 884	....	.....	.....	.....	...
1890	674 535	....	.....	.....	.....	...
1895	781 594	90.6	70.82	.....	.....	...
1900	897 635	111.4	100.00	.....	.....	...
1905	975 077	125.3	122.13	116.08	117.96	137
1906	.....	122.9	122.83	117.03	118.17	128
1907	.....	126.8	129.88	123.67	124.81	147
1908	.....	125.1	131.27	124.92	126.07	141
1909	.....	116.5	124.89	119.12	119.84	130
1910	1 098 048	107.6	118.22	112.09	112.96	125
1911	.....	103.1	116.09	110.00	111.37	120
1912	.....	105.5	121.59	116.23	117.39	139
1913	.....	92.7	109.45	103.85	104.91	116
1914	.....	93.5	112.88	107.04	108.45	124
1915	1 234 790	87.1	107.52	101.94	103.13	110
1916	.....	90.1	112.28	106.34	107.46	113
1917	.....	92.4	116.23	110.03	111.36	122
1918	.....	107.5	136.33	129.76	131.28	162
1919	.....	99.6	127.39	120.59	121.81	131
1920	1 290 651	104.1	134.40	127.27	128.65	149
1921	.....	95	124.69	117.41	118.74	128
1922	.....	95	126.74	119.27	120.74	129
1923	.....	98.1	133.04	125.24	127.25	136
1924	.....	96	132.18	124.10	126.84	135
1925	1 398 702	97.6	136.52	128.35	134.50	144
1926	.....	97.8	138.65	130.18	134.60	141
1927	.....	98.2	141.15	132.49	136.18	141
1928	.....	99.1	144.44	135.69	139.59	146
1929	.....	99.1	146.51	137.04	142.27	150
1930	1 498 253	97.5	146.11	136.42	143.11	158
1931	.....	95	144.57	134.78	138.09	145
1932	.....	89	137.68	127.99	131.48	139
1933	.....	87	134.39	125.07	128.64	136
1934*	.....	.....	.....	137	.....	...

*3 months.

TABLE 16.— CAPACITY OF PRINCIPAL RESERVOIRS AND AQUEDUCTS, METROPOLITAN WATER SUPPLY.

	Capacity Mil. Gal.		Area — acres.	Approximate grade-line capacity — m.g.d.	Length — miles.
	Total including flashboards as ordinarily operated.	Available above gate sill.			
Quabbin Reservoir	415 000	405 000	25 000		
Quabbin Aqueduct				340	24.61
Wachusett Reservoir	67 000	57 000	4 135		
Wachusett Aqueduct				350	11.95
Sudbury Reservoir	7 250	6 000	1 292		
Weston Aqueduct				300*	
Weston Reservoir	270	200	60		13.42
Sudbury Basin No. 3	1 180	1 000	250		
Sudbury Aqueduct				100	17.38
Chestnut Hill Reservoir	....	690	123		
Spot Pond	....	1 790	309		

* Only about half of this until 2nd and 3d pipes are added in the Sudbury River and Happy Hollow siphons.

The above table does not include Cochituate Lake and Aqueduct and the following reservoirs in the Southern Sudbury area: Whitehall, Hopkinton, Ashland and Sudbury Basins Nos. 1 and 2.

PART V.

ORIGIN OF THE NAME QUABBIN.*

The new reservoir is named for the old Indian chief Quabbin who stood beneath the great oak on the broad slope of what is now Quabbin Hill, across the river from the little village of Enfield, and looked north as far as the eye could see along broad flat valleys speckled with mirror ponds and dotted with rocky peaks. These valleys had furnished good fishing and hunting for his tribe, the Nipmucks, for untold generations before the first white settlers came from Springfield. They were forerunners of the white man's thirsty civilization which would one day dam the narrow outlet and create a lake of greater extent than the old chief had ever seen. Tribes of Nipmucks, Nipmugs or Nipnets, as they were sometimes called, fished and hunted over all this section of central Massachusetts long before the coming of the white man to America. The Indian name implies that the tribes dwelt in "fresh-water fishing places" or in "well watered" valleys. Standing on Quabbin Hill overlooking the village of Enfield nestled in the valley five hundred feet below and viewing the vast and quiet expanse of the river

* In preparing the following descriptive notes, I have depended largely upon researches by George W. Coffin, one of the engineering force.

valley that is lost in the haze to the north, one can easily believe the historians who tell us these Indians were a peaceful people, unlike the warlike Mohawks on the west and Narragansetts on the east.

Historians tell us that the name Quabbin is probably the Anglicized form of Ninequabin, Naniquabin or possibly Wabequabban, the prefix Nine, or Nani, signifying chief. Be that as it may, local tradition gives us the spelling Quabbin and associates the old chief with the towering hill and the general locality both of which bear his name. It was not the custom of the red man to name places after persons, either living or dead. This was undoubtedly done by early white settlers of the region although it is true that the name was used by later remnants of the earlier Indian tribes.

The first white men to view the valley were probably a few adventurous spirits from the trading post established about the middle of the 17th century on the Connecticut River where Springfield is now located. The first real settlers to homestead in the valley obtained their grants to "a tract of land lying at a place called Quabbin" as a reward for services rendered either by themselves or their fathers to Massachusetts Bay Province as soldiers in King Phillip's War, particularly in the defence of Fort Narragansett in 1675. In 1738, a plat of 18 079 acres of land was laid out in the Swift River Valley and confirmed by the Great and General Court as Narragansett Township No. 4, which was the "area called Quabbin". This tract was "bounded Eastwardly on Lambs Town," (Colonel Lamb and his company were established in what is now Hardwick), "Southwardly on Reads Land," (Read apparently had a prior grant in what is now Ware), "Westwardly partly on Equivalent Land" (Governor Saltonstall's heirs *et al.* had been granted an equivalent of land in what is now Prescott), "and partly on Salem new Town, Northwardly partly on Salem Town and partly on Whites Town" (apparently to the north, near New Salem). The quotation is from Chapter 241 of the Acts and Resolves of the Province of Massachusetts Bay, 1738. In 1743 Pelham was incorporated and bounded "easterly on a tract called Quabbin," being the tract previously granted as Narragansett Township No. 4, and including what is now Greenwich, Enfield and part of Dana. This entire tract was incorporated in 1754 as the town of Greenwich. In 1816 the southerly part of this area was set off as the town of Enfield.

Apparently the early settlers who received the land as a reward for fighting King Phillip did not appreciate the "well watered" plain as did the Nipmucks who were well satisfied with the fishing. They were more interested in well drained upland areas where the maize and tobacco would grow without too much attention. Upon their complaint to the legislators, they were awarded additional "equivalent land" west of the Connecticut River near Williamsburg, which accounts for the name "Quabbin 2nd Grant" on early maps in that vicinity.

The remnants of Chief Quabbin's followers gradually dwindled as the white man overran their lands. Every waterfall furnished the site for a

grist mill and many thriving communities dotted hill and plain in the middle of the last century. It was on Prescott hill that Daniel Shay drilled his men for "Shay's Rebellion." Following the Civil War the great cities along the seaboard and the larger rivers continued their growth at the expense of rural communities, and began to suck the life blood from these tiny villages in the Swift River Valley.

Today there is a feverish activity of steam shovels and concrete mixers in the valley, but soon these evidences of the white man's occupation will disappear. The water will slowly creep up the sides of the Prescott peninsula where the deer will again roam undisturbed and the "well watered" valley extending north between rocky island peaks will be a fitting happy hunting ground for old Chief Quabbin.

Conclusion.

Many interesting features of the work have of necessity been omitted from this paper. Such for example are many details of the design and methods of construction, methods used by the engineering force in real estate and topographic survey work, and miscellaneous public relations.

PART VI.

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Discussion.

Frank E. Winsor.* The author has covered the subject so thoroughly as to leave little opportunity for either criticism or constructive additions to the paper. It may be of interest, however, to consider in further detail some subjects covered by the paper.

Subsurface Investigations. The subsurface investigations for the Quabbin Aqueduct fixed the line of the tunnel as shown upon the plans which have been presented. A straight line from Wachusett Reservoir at Oakdale to the nearest point of Quabbin Reservoir would have been slightly shorter than the line adopted, and a straight line from Wachusett Reservoir at Oakdale to a point much nearer the southern end of Quabbin Reservoir would have been preferable from the standpoint of taking water from a greater distance from the entering points of the principal tributary streams, thus securing the benefit of a longer time in storage. It was necessary, however, that the line of the tunnel should pass under the Ware River within the limits prescribed by the legislature which necessitated a location considerably north of the straight line just referred to.

At the Wachusett end of the tunnel, where it is expected that a power plant may later be installed, there was a choice of building the aqueduct at the level of Wachusett Reservoir in which case it would have been necessary to provide for a connection to the power plant from the aqueduct under a 135-ft. head (with a maximum possible head at times of maximum diversion from the Ware of about 175 ft.) or, as was done, depress the aqueduct and run it under pressure, bringing the flow into the reservoir upward through a vertical shaft to which the power plant will connect. The subsurface investigations showed the high ground near the Wachusett outlet to be merely a glacial deposit but showed rock near the ground surface at Shaft 1 and coupled with estimates of cost indicated clearly that the method adopted is preferable.

Although the possibility of the power development alone might not have justified putting the entire tunnel through to Wachusett Reservoir under the full head of Quabbin Reservoir, this was the most practicable solution of the problem of diverting the Ware and Quinapoxet flows westward to Quabbin Reservoir or eastward to Wachusett Reservoir at will.

Investigations for the location of Shaft 2, two miles west of Shaft 1, indicated the desirability of an angle at this point both to secure rock near the surface for the shaft location and also to provide for the possible subsequent diversion of the Quinapoxet River into the aqueduct, from which point it may flow either eastward into Wachusett Reservoir or westward and be stored in Quabbin Reservoir. From Shaft 2 to Shaft 8, a distance of 11½ miles, the aqueduct line is substantially straight. West of Shaft 8 the line originally investigated with an angle at the shaft encountered rock

*Chief Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

near the Ware River downstream from the shaft so low as to indicate the probability that earth would be encountered at the tunnel grade. For this reason the tunnel line from the east was extended straight through Shaft 8 and an angle made on the north side of the river, thus avoiding the difficulties and dangers of soft-ground tunnelling.

Investigations of the valley of Muddy Brook, where Shaft 11 is located, led to the adoption of a line considerably further upstream than originally contemplated in order to insure sufficient rock cover for the tunnel roof. Between Shaft 11 and the reservoir a considerable length of the line was obscure as to depth of rock, and while the geologists were of the opinion that there would be sufficient rock cover, it was deemed desirable to supplement the diamond drill borings by further subsurface investigations which for purposes of economy and to save time were made for a distance of about ½-mile by electrical methods.

The entire 25 miles of tunnel, as has been stated in the paper, are now driven, and no heavy ground was encountered at any point, the timbered sections, less than 0.4-mile in length, being for the purpose of insuring safety of workmen and to provide slightly increased thickness of masonry where the rock encountered was blocky or showed instability.

Another notable feature was the economy effected by the use of Gunite to coat the rock roof in portions of the tunnel somewhat over two miles in length where the air slacking of seams caused the loosening and falling of rock. The necessity of enlarging and timbering the tunnel was thus avoided.

The legislation required that the Commission shall collect and publish in its reports such information as to the geology of the region as will be of value in connection with the geological history of the State. Very careful geological investigations were made in advance, and records have been kept during construction. Samples of the rock encountered have been permanently preserved and will be available for future study. This 25-mile cross-section forms a unique permanent record of the geology of Central Massachusetts. It is expected that a complete geological report will be printed at an early date.

The work of the geologist has been of prime importance not only on the location of the tunnel but on the investigations of the main dam, the dike and the highways which have been built or are under construction. Studies both of the underlying rock and of the overburden have been of great importance as predicating in advance the location and character of rock to be encountered, the availability of excavated materials for masonry construction and the character of materials, such as gravel, and stone ballast, needed for the building of highways, for embankments of the main dam and dike, and other constructions.

Quabbin Aqueduct Operation. After the Quabbin Reservoir is completed the hydraulic grade in the Quabbin Aqueduct will stand over 100 ft. above the tunnel arch, and the pressure will be carried directly by the rock as the tunnel lining is well provided with vent holes so located as to prevent

undue pressure coming upon the lining either when the tunnel is under pressure or when it is empty. The ground water will be higher than the maximum hydraulic grade in the aqueduct except for some 3 000 ft. at the Wachusett end. For 500 ft. at this end the tunnel is circular and, together with the shaft, grouted solidly against the rock. Should there be slight external leakage from the tunnel at this end, the leakage will flow directly into Wachusett Reservoir.

The Quabbin Aqueduct is unique in the number of functions which it performs. Water taken from the Ware River at Shaft 8 may be conveyed eastward into Wachusett Reservoir or westward into Quabbin Reservoir. Due to the high elevation of the Ware River, no storage is required on the Ware watershed as the tunnel is of sufficient size to take the highest flood flows, although it has been built no larger than necessary to carry the required flow from Quabbin Reservoir to Wachusett Reservoir. Provision may easily be made so that waters from the Quinapoxet River, a tributary of the Wachusett Reservoir, having a drainage area of 44 sq. miles, may be diverted into the tunnel at Shaft 2 and similarly carried either to Wachusett Reservoir or Quabbin Reservoir.

Quabbin Reservoir. This reservoir is materially larger than would ordinarily be designed for a drainage area of its size. It is an unusual natural basin, storing water at the elevation adopted to an average depth of over 50 ft., and the expense of building it at a lower elevation would not be greatly reduced. The reservoir stores the flow not only from its own drainage area of 186 sq. miles but is available for storage of the flow from 98 sq. miles of the Ware and 44 sq. miles of the Quinapoxet. It is an ideal basin both as to shape and as to character of the soil, which is almost uniformly sandy with negligible areas of swamp. It is probable that in the far distant future when and if additional water is needed for the Metropolitan District it may be obtained by connecting other sources to Quabbin Reservoir and storing the waters therein. The portions of the Ware and Swift drainage areas to be used for water supply are very sparsely populated and with the long-time storage provided in Quabbin and Wachusett reservoirs will furnish a supply of low color and of excellent quality.

Frank E. Fahlquist.* Construction of the Quabbin Aqueduct Tunnel and investigations at the dam sites have contributed greatly toward a better understanding of the geology of Central Massachusetts. This additional knowledge is confined not only to the purely scientific aspect of the subject, but as well to the applied side especially as it relates to the several rock types differing widely in character and behavior. A brief discussion of the geology of the area, in relation to this project, may be presented under two headings: first, the geology of the topographic features and surface deposits, and second, the geology of the underlying rock formations.

Geology of Topographic Features and Surface Deposits. The topo-

*Assistant Engineer, Metropolitan District Water Supply Commission, Boston, Mass.

graphical forms and surface deposits of this area, as throughout New England, are the results of several geologic processes, among which is that of glaciation. The widespread deposits of sand, gravel, boulders, and clayey material are the products of glacial abrasion and deposition. Two principal types of deposits are easily recognized. One of these, sometimes designated by the engineer as hardpan and by the geologist as unmodified drift, is composed largely of fine-grained particles and larger rock fragments such as cobbles and boulders. The other type, known as modified glacial drift, consists of sand and gravel showing evidences of having been deposited by the aid of water. In general, the former deposits are found in the higher elevations, whereas the latter are found in the valleys.

Except for a few scattered outcrops the character of the underlying rock formations has been obscured by occurrence of this drift material. The unmodified drift varies from a foot or less to about thirty feet in thickness, although in some instanees this may be greater. The modified drift has a greater range of thickness and in some sections has been deposited to a depth exceeding 175 ft. It was found helpful in the location studies for the tunnel to locate the approximate limits of these two principal types, especially the unmodified drift, as this contributed some information at least to the problem of obtaining a rock profile. Each of these two kinds of material will have a part in the construction of the two dams in the Swift River and Beaver Brook valleys. Core material for each of these dams may be obtained from the unmodified glacial drift areas nearby, and heavier and more pervious material from the modified drift areas, within the lower parts of the valleys.

The underlying rock topography in many instances is quite different from that of the ground surface, and the only means of obtaining definite information for locating the tunnel was by some method of subsurface exploration. An extensive program of investigations by core borings was carried out, the borings in general being concentrated at recognized critical areas, chiefly in the Quinapoxet River, Ware-Prince River and Muddy Brook valleys. A location different from that originally planned was adopted in all these valleys, in order to be sure that the tunnel would be excavated in rock for its entire length. In the Quinapoxet valley the tunnel was depressed about 200 ft., but in the Ware-Prince and Muddy Brook valleys its location was changed in plan in order to pass under these valley floors at points more favorable for obtaining a safe rock cover. This, as closely as could be determined, amounts to between 50 and 60 ft., and in excavating through these two sections the conditions encountered were as favorable as they would have been with a much greater rock cover.

Removal of the glacial drift material throughout this area would reveal a drainage pattern considerably different from that now present. For instance many of the present surface divides are the result of accumulation and dumping of glacial drift material and have no counterpart in the underlying rock floor. This is the case of the Swift River divide in Muddy

Brook valley, where profiles developed from borings indicate a large continuous rock valley, which at one time was part of a much younger Swift River system. It is likewise true of conditions in Beaver Brook valley, where one of the dams for the Quabbin Reservoir is being constructed.

Geology of Underlying Rock Formations. All the rocks penetrated by the tunnel are of crystalline character and may be divided into the two classifications of granites and schists. The former occur in three distinct belts, and have been designated as the Fitchburg, Hardwick, and Monson granites. They occupy about eight miles of the total length of tunnel. The Fitchburg and Monson formations are typical light-colored granites of average hardness. The Fitchburg granite contains masses of a more micaceous rock, which in general is slightly softer than the granite but in some instances harder. These included rocks are remnants of an older formation into which the Fitchburg granite was intruded. A darker and harder igneous rock, or diorite, occurs as beds or broad bands in the Monson granite. The Hardwick granite is somewhat different from the Fitchburg and Monson granites in that it is much darker, contains much more mica than average granite, and is slightly harder. This formation also contains beds of a softer micaceous schist.

The schists or laminated rocks, which occupy about seventeen miles of the belt traversed by the tunnel are divided into five formations. Although they are now highly crystalline, they originally constituted a thick series of sedimentary rocks consisting of sandstone and shale. These primary differences may now be recognized by changes in the mineral content from bed to bed. What was originally sandstone now appears as quartzite or quartz schist. Portions of these formations which were shale have been changed into schist containing a high percentage of mica and having very fine laminations. Hence the term mica-schist has been applied to three of these formations, and quartz schist and quartzite to the other two.

The strength and hardness of the schist rocks has been greatly increased by the addition of granitic material. These igneous intrusions have permeated the rocks on a grand scale, so that it is often impossible to recognize whether some portions have originated as true igneous rocks, or as sediments which later have been transformed by being soaked with granitic constituents. Igneous intrusions generally occur as streaks, bands, and dikes, forming an interlocking network. In one particular mica-schist formation, about twenty-five per cent. of the present bulk is estimated to have been added. In general, the schist rocks are somewhat softer than the granites and of these the mica schists are the softest.

Two diabase or trap-rock dikes were penetrated by the tunnel, one being about 275 ft. wide and the other about 100 ft. These have been traced for several miles, in scattered outcrops and appear to cut across this area in a nearly parallel northeasterly direction. In the tunnel they are about ten miles apart.

The structure, or inclination of the bedding in the schists and the foliation or grain of the granites are surprisingly simple, although their interpretation is difficult. The geologic section of the tunnel shows a wide belt of only slightly inclined schist rocks flanked by rocks having a somewhat steeper inclination, with scattered areas where the structure is vertical or nearly so. The slightly inclined bedding of the rocks in the tunnel adjacent to Shaft 4 and Shaft 9 is one of the contributary causes of excessive rock scaling following excavation, which however was successfully remedied by Guniting. One of the geologic surprises in the construction of this project is the absence of crushed or faulted zones requiring timber support. In tunnelling through several rock formations over such a great distance, the occurrence of such weak zones would normally be expected. Faulting and extreme crushing has occurred at many points, but this happened before the formations were subjected to igneous invasions. The injection of granitic material into these zones replaced the older and weaker rock with one which was stronger and more crystalline. Very little permanent timbering was required, and most of this was necessary because of excessive scaling, due to a combination of conditions, including air slacking, thin bedding and the nearly horizontal position of the strata. At one point, however, in the Monson granite near Shaft 12, the tunnel was excavated through a zone of crushed and altered rock. In this section, about 120 ft. long, the clayey material encountered had a tendency to move and develop pressures.

ROBERT SPURR WESTON.* Mr. Kennison has honored me with a request to discuss his paper. However, the paper is so complete, especially regarding the general design and methods of construction, and has been so well discussed by Mr. Winsor that any further elaboration of its subject matter would be superfluous. I am, however, much interested in the quality of the water which the present works has supplied and in estimating that which will be supplied by the new works, also in the probable effect of the combination of the new with the old upon the water drawn from the Metropolitan taps in future years.

In 1922, when the Joint Board presented its report to the Massachusetts Legislature (House Document 1550), it was estimated that the consumption by the District in 1930 would be 168.5 m.g.d. Owing to economic distress, the actual consumption in that year was 143.1 m.g.d., and in 1933, the consumption had fallen to 131.48 m.g.d. The normal increase in consumption has been temporarily arrested, largely due to a decrease in the industrial use of water. Therefore, it is difficult to predict when and to what degree it will return, or what the more distant rate of increase will be. From past experience, it may be expected that the rate of increase will for a time be increased after industrial recovery takes place.

The rate of consumption is important from a water quality standpoint because storage is and will be the chief method of purification and the one most depended upon. The degree of purification by storage is largely

*Consulting Engineer, Boston, Mass.

dependent upon the period of storage and the latter upon the rate of consumption.

In the future, storage will be provided in the Quabbin, Wachusett and Sudbury reservoirs, as well as in the distributing reservoirs of the system, like Weston and Chestnut Hill reservoirs and Spot Pond.

The Quabbin Reservoir will be an ideal one. The reservoir is deep; its catchment area has a sparse and declining population; the natural soil is low in organic matter, and the period of storage long. With a yield of 210 m.g.d., and a storage capacity of 415 000 m.g., its storage ratio* will be 5.49, which is equivalent to about 5½ years. This may be compared with a storage ratio of 1.57, equivalent to over 1.5 years, for the Wachusett Reservoir. Storage of the Swift River water alone for 6.6 years, which would be the average condition were no water diverted into the Quabbin Reservoir from the Ware, or storage in the Quabbin Reservoir of the combined Ware and Swift waters for 5.5 years, will effect a high degree of purification. Although there may be some growths of organisms for a few years, while the reservoir is being conditioned, the reservoir will afterwards discharge a nearly clear and colorless water and one as safe as storage alone can produce.

By and large, it is appearance, particularly the color, which is the best criterion for judging this water. The color of the water which will be diverted from the Ware River, is about 55 p.p.m., the color of the Swift River water which will be stored in the reservoir is about 35 p.p.m., or slightly lower than that now entering the Wachusett Reservoir. But the period of storage is so long that a reduction to a color below 2 p.p.m. may be confidently expected,— just such a reduction as occurs in some of the lakes and ponds within the Chicopee water-shed which provide equally long periods of storage. So long is the period of storage that it will make little difference whether or not the natural runoff of the Swift catchment area or the more highly colored diversion water from the Ware, or the somewhat polluted Quinapoxet water is stored. It will, however, make considerable difference in the color of the water discharged from the Wachusett Reservoir whether it is fed by the runoff from its own catchment area, by the flood waters of the Ware, or the stored waters from the Quabbin Reservoir. The Wachusett feeders discharge a water having a color of about 38 p.p.m., which, strangely enough, is but slightly higher than that of the Swift River water. The color of the diverted Ware is estimated at 55, and the stored Quabbin water at 2 p.p.m.

Notwithstanding the construction of a sewer to care for the pollution within the Ware watershed and the demolition of buildings on property purchased for protection, the Ware River at Coldbrook can never be free from pollution, although it may be reduced to a slight amount. It may be guessed, therefore, that whenever practicable, the management will store the Ware water with the Swift water in the Quabbin Reservoir, and after

*The storage ratio is the quotient of storage capacity divided by annual yield.

as long a period of storage as can be secured in practice, will let it down into the Wachusett, thus feeding the latter with the highest practicably obtainable grade of water. With the Wachusett, material pollution begins. The city of Worcester is overflowing into the valley of the Quinapoxet, one of the feeders. Furthermore, this river receives considerable amounts of industrial wastes. The town of West Boylston at the head of the reservoir is growing, and in summer thousands of motorists pass daily. The Stillwater, the other main feeder, is also somewhat polluted. So that although the Commission may do much to control conditions, and although as Mr.

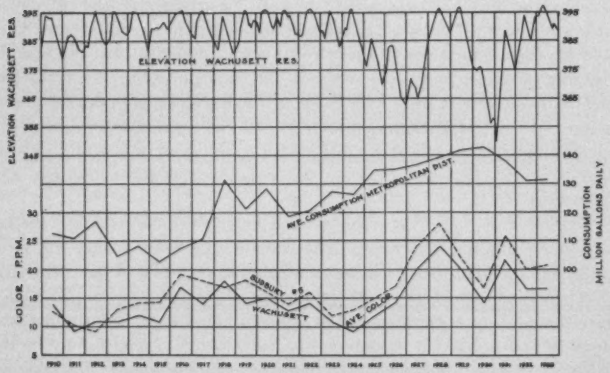

FIG. 20.— ELEVATIONS OF WACHUSETT RESERVOIR, WATER CONSUMPTION OF METROPOLITAN DISTRICT, AND COLORS AT WACHUSETT AND SUDBURY RESERVOIR DAMS, 1910–1933.

Winsor has stated, a part of the Quinapoxet may be diverted to Quabbin Reservoir and there purified by storage before reaching the Wachusett, pollution must be reckoned with.

Here storage again comes into play. The Wachusett stores 67 090 m.g. when full. With its mean annual runoff of 42 655 m.g. this is equivalent to a storage ratio of 1.57, equivalent to about 573 days. This period will effect an average reduction in color of at least 65 per cent. and render the water safe. Soon after the Wachusett was placed in service, long periods of storage obtained, and colors of delivered water were low. As the consumption increased, both period of storage and color decreased. This is shown in the following table and diagram (Fig. 20), giving the colors of the delivered water for various years.

The available catchment area of the Wachusett was formerly 108.8 sq. miles, but the new legislation on behalf of Worcester has reduced that which can be devoted to the Metropolitan District to 88 sq. miles. Conse-

AVERAGE COLORS OF WATER AT WACHUSETT AND SUDBURY DAMS.
(From Reports of the Metropolitan District Commission.)

Year.	ELEVATION OF WACHUSETT RESERVOIR.*		AVERAGE COLOR — p.p.m.		Total Consumption m.g.d.
	Maximal	Minimal†	Clinton Dam	Sudbury Dam	
1910	394.25	378.52	14	13	112.96
1911	387.41	381.3	9	10	111.37
1912	395.36	383.71	11	9	117.39
1913	395.38	386.5	11	13	104.91
1914	395.43	381.17	12	14	108.45
1915	392.42	389.0	11	14	103.13
1916	395.75	384.97	17	19	107.46
1917	395.55	382.5	14	18	111.36
1918	393.50	380.77	18	17	131.28
1919	395.93	387.53	14	18	121.81
1920	396.01	388.3	15	16	128.65
1921	396.06	386.42	13	14	118.74
1922	396.24	386.5	14	16	120.74
1923	395.65	381.42	11	12	127.25
1924	396.39	376.0	9	13	126.84
1925	386.19	369.60	12	15	134.50
1926	383.25	362.76	14	17	134.60
1927	385.44	365.1	20	24	136.18
1928	396.61	388.7	24	28	139.59
1929	396.50	373.5	20	22	142.27
1930	376.55	350.00	14	17	143.11
1931	388.79	374.66	22	26	138.09
1932	394.40	384.1	17	20	130.92
1933	396.55	387.81	..	..	131.48

*Elevation of high water — 395 ft.
†This is the minimal elevation either in the fall of the year or early in the following year.

quently the storage ratio under the new conditions is increased to 1.95, which is equivalent to about 710 days, assuming no addition from the new sources. It is probable, however, that Worcester will not require additional water for many years. Consequently this addition to the storage ratio need not be considered for some time.

The runoff from this area is inferior to the stored water from the West, although lower in color than the unstored Ware River water at Coldbrook. All of it, however, must be stored and used. It cannot be replaced by the new water.

Assuming a draft of 150 m.g.d. from Wachusett, the colors of the entering water are estimated at 41 p.p.m. with the unstored Ware being let down, or 28 p.p.m. with the stored Quabbin water being let down, and the result-

ing colors of delivered water at 18 and 12 p.p.m., respectively. Naturally, if the Quinapoxet is diverted through Shaft 2, stored in the Quabbin Reservoir and then let down again, the color and quality of both entering and delivered waters will be somewhat improved. The excellence of the discharge of the Quabbin Reservoir would still be maintained.

When 200 m.g.d. is supplied to the Wachusett Aqueduct with the Wachusett catchment area reduced to 88 sq. miles, and stored water let down from the Quabbin Reservoir, the color of the entering water may be reduced to about 19 p.p.m., and the color of that at the Wachusett intake to about 10 p.p.m. Further increments of stored water from the Quabbin Reservoir may reduce the entering color to below 15, perhaps to 13 p.p.m., but with reduced storage in the Wachusett at the higher rates of consumption, the color of the water at the intake of the Wachusett Aqueduct will rarely go below 8 p.p.m. which is, however, lower than it has ever averaged.

Insofar as color and bacterial contents go, the water in the Wachusett Aqueduct should be of high grade. Whether or not the reduction in storage in the Wachusett Reservoir to considerably less than a year's flow will promote the growth of micro-organisms, is difficult to predict. It may be possible that the smaller amount of organic matter in the Quabbin water will compensate for the reduced effects of storage in the older reservoir. If not, high numbers of microscopic organisms and consequent disagreeable odors and tastes may occur for short periods of the year as they have before.

With the Sudbury Reservoir, new problems arise. Located in a growing suburban zone, it is undoubtedly true that the Wachusett Aqueduct water will deteriorate in passing through it. At present, considerable trouble is experienced with growths of organisms in this reservoir, and the water now leaving it is somewhat inferior to that leaving the Wachusett Reservoir, particularly in appearance, which is the consumer's chief criterion. The Sudbury Reservoir stores but 7 253 m.g., and has a mean yield of but 22 m.g.d. With 120 m.g.d. passing through it, and assuming complete displacement, which, of course, cannot be obtained under the conditions, the period of storage is but sixty days, while with 300 m.g.d. passing through it, it will be but fourteen days. Experience has shown that reservoirs providing these short periods of storage usually suffer from growths of microscopic organisms. Furthermore, the arms of the reservoir, which are outside the current of water flowing through from the Southboro channel to the intake at the dam, will at times produce excessive growths of organisms, and these may seed the better water flowing through and promote growths not only near the dam, but in Weston Reservoir, Chestnut Hill Reservoir, Spot Pond and elsewhere. Treatment with copper sulphate, however, may control these growths sufficiently.

The engineers who have reported upon the general problem in the past, have all recommended the ultimate filtration of the supply, and some are agreed that a purification plant could well be located in Framingham, along the line of the Weston Aqueduct. This opinion is in accord with the

fundamental sanitary engineering principle, namely, to get as good a natural water as you can, and then purify it as best you can.

It is also possible, as Mr. Hazen has suggested, to construct a new distributing tunnel from the Wachusett Reservoir to the district, which would discharge a water which would require nothing more than disinfection to make the water wholesome and agreeable. This plan, of course, would leave the Sudbury sources unpurified, but it would of course cost far less to purify these alone. Furthermore the water of the South Sudbury now wasted could be included in the plan. While disinfection of the water from the Sudbury sources would make the water safe, it is also probable that the treated water would not meet esthetic standards, especially when compared with water supplied directly from the Wachusett Reservoir. It is also probable that the addition of treatment with copper sulphate would fail to make the water acceptable.

When this problem of appearance must be met, by filtration or otherwise, is difficult to estimate. Its solution may be forced by the water let down from the Quabbin Reservoir before the latter is aged, or by a reduced period of storage in the Sudbury Reservoir, both tending to increase odors and tastes. On the other hand, treatment with copper sulphate may tide over the initial difficulties, or long storage in Quabbin Reservoir may compensate for reduced storage in the Sudbury Reservoir, and the necessity for purification or a distributing tunnel may be postponed.

Various cities in the United States, for example New York, Cincinnati and Kansas City, have found that uncontaminated water is often recontaminated in passing through open reservoirs. Connected with one supply supervised by the writer's firm are two reservoirs,— one open and one covered, on similar hills, but both supplied from the same source. During 1933, intestinal bacteria (*Bact. coli*) were found in the water supplied from the open reservoir for over 8 per cent. of the time, while gas-forming bacteria were present seven months out of twelve, as compared with finding *Bact. coli* in the water from the covered reservoir less than 2 per cent. of the time, and gas-forming bacteria for only one month out of twelve.

Neither upland storage nor filtration will prevent accidental or vicious contamination of distributing reservoirs by birds and animals, including the human animal, and it may be found judicious to cover some of the smaller reservoirs in the Metropolitan District, although it will probably be more practicable, as at present, to depend upon disinfection with chlorine for protection against the accidental contamination of the larger ones.

Massachusetts may look with pride upon the works which have supplied and will supply Boston, from the Jamaica Pond works with its wooden pipe line, to the Quabbin Reservoir and connections which Mr. Kennison has so interestingly described. While many of us may not be active when the Quabbin water first reaches the local taps, we can be thankful for the good judgment of the various water-supply commissions, for the

careful planning, past and present, which has characterized the Metropolitan Works, and for the kind of engineers under whose direction the works have been built and operated so well.

ARTHUR D. WESTON.* Mr. Kennison's paper covers clearly and concisely the works constructed by the Metropolitan District Water Supply

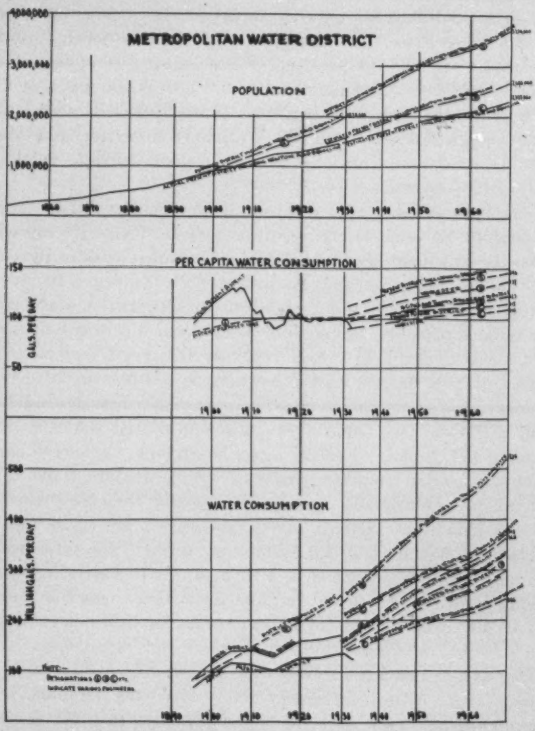

FIG. 21.— POPULATION AND WATER CONSUMPTION OF METROPOLITAN WATER DISTRICT.

Commission and should prove to be of considerable value not only to the members of this Association but also to the public authorities and those engaged in water-works practice.

With the completion of the tunnel from Wachusett Reservoir to the Ware River at Coldbrook, whereby water has already been diverted to

*Chief Engineer, Massachusetts Department of Public Health, Boston, Mass.

supplement the water supply of the Metropolitan Water District, it may be of interest to make comparisons of the various estimates concerning the water supply needs and resources of this district and the contemplated and actual effect of such diversion on the quality of the water of Wachusett Reservoir.

Population. In discussing ways and means of increasing the water supply of the Metropolitan cities and towns, F. W. Stearns, as chief engineer of the then State Board of Health, estimated in his report published in February, 1895 as House Document No. 500 of that year, that a population of 1 978 000 would be reached in the year 1930 by the cities and towns now included in the Metropolitan Water District plus the city of Newton and the town of Brookline. The district failed to grow as rapidly as estimated by Stearns and the actual population of this group according to the census of 1930 was 1 498 253.

Water Consumption. Stearns estimated that the per capita water consumption in 1930 would equal 100 g.p.d., but extensive metering of services was necessary to reduce former high consumption to this figure by 1930. Goodnough in his report to the Joint Board, as published in House Document No. 1550 of the year 1922, and Metcalf in connection with the report of the Metropolitan Water Supply Investigation Commission, published as House Document No. 900 of the year 1926, starting with actual water consumptions estimated that the per capita use of water would increase until in the year 1970 it would reach 146 g.p.d. (Goodnough) and 115 g.p.d. (Metcalf). The estimated future water consumption from Metropolitan sources was estimated by Goodnough and Metcalf on various assumptions as to the municipalities which might enter the district and the degree to which they might continue to use their own sources; therefore these estimates may not be compared on the same basis.

As things have turned out, it looks as if, during the next ten years at least, the estimates of Metcalf, both as regards water consumption and population, will resemble more closely the actual conditions. The various estimates made in regard to the future water supply of the Metropolitan Water District are shown in Fig. 21, those of 1895 being marked A, those in relation to the 1922 study as B, and those of the 1926 report as C.

Drainage Areas and Yields. A comparison of the various ways and means proposed for securing an additional water supply for the Metropolitan Water District appears to the writer to be particularly interesting. The 1895 study under the direction of F. W. Stearns assumed that about 173 m.g.d. of water would be available from the Sudbury River, Lake Cochituate and Wachusett drainage areas, that additional water from the South Sudbury and Assabet River drainage areas would bring the safe yield to 201 m.g.d. and when necessary by diverting the Ware River at Coldbrook some 71 m.g.d. additional would be secured and finally, by utilizing the Swift River and Lower Ware River drainage areas for an additional 200 m.g.d., a total of 472 m.g.d. would be available. Compared with

these estimates, the Joint Board in 1922 proposed the development of the North Sudbury sources with Wachusett and the Ware River at Coldbrook to yield about 160 m.g.d. and the subsequent development of the Swift River by utilizing the flood flows of this river over 1.2 c.f.s. per sq. mile and the construction of a large reservoir which would bring up the safe yield of the sources to 233 m.g.d. This report also contemplated later development of the flood flows of the Millers River, when necessary, and possibly of the lower Ware River watershed, making a safe yield from all sources of about 352 m.g.d. It will be remembered that the investigating

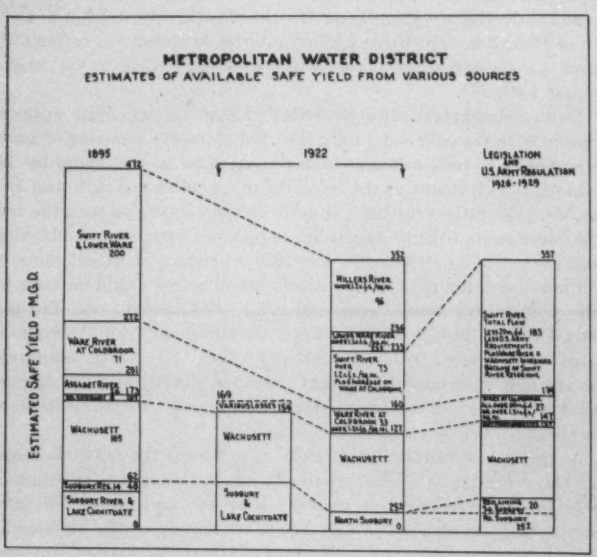

FIG. 22.

commission, known as the Gow Commission, in the report of 1926, recommended the development of sources of water supply chiefly in the easterly part of the state and, therefore, these estimates cannot be compared with those of 1895 and 1922.

After consideration of the recommendations of the Metropolitan Water Supply Investigating Commission and those of the Joint Board in 1922, legislation was enacted by the Commonwealth and later regulations were adopted by the U. S. Army. In accordance with this legislation and the U. S. Army regulations the present works were designed for a safe yield of the various sources as follows:

North Sudbury area 25 m.g.d.
South Sudbury area 20 m.g.d.
Wachusett drainage area and purchase from Worcester 102 m.g.d.
Development of Ware River at Coldbrook 27 m.g.d.
Flood flows from Swift River, additional diversion of the Ware River at Coldbrook 183 m.g.d.

The total yield is 357 m.g.d., which is about the same in gallons per day as proposed in 1922. A comparison of these various developments as proposed is shown in Fig. 22.

A comparison of the future estimated requirements of the Metropolitan Water Supply District with estimates of the safe yield of the sources

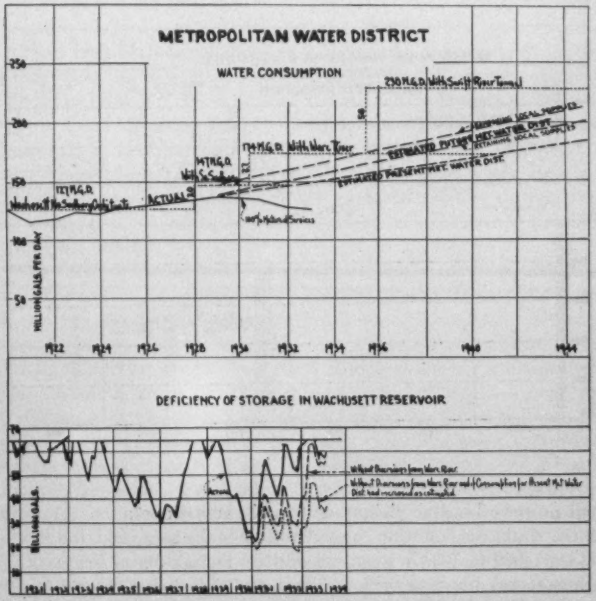

FIG. 23.— WATER CONSUMPTION AND DEFICIENCY OF STORAGE IN WACHUSETT RESERVOIR.

to be developed would indicate that Wachusett, North Sudbury and Lake Cochituate, plus the development of the South Sudbury, would have been sufficient up to 1931 if the use of water from Metropolitan sources had increased as estimated and if those municipalities which it was expected would enter the district had entered and continued to use their local sources. If membership in the district had increased as was contemplated

and these new members had not continued to maintain their local sources of supply, the safe yield probably would have been equalled by 1929. Actually, because of decreased water consumption, the safe yield of this supply was sufficient for the Metropolitan Water District through 1931 and would have been sufficient under the actual yields obtained even without the Ware River. This condition, which is shown in Fig. 23, is particularly interesting upon analysis when studied together with the depletion of storage of the sources of water supply as shown on the bottom of this diagram.

It should be remembered that in estimating the safe yield of the present sources of water supply the dry period from 1908 to 1915, inclusive, was used as a basis. This period probably is the longest dry period of that

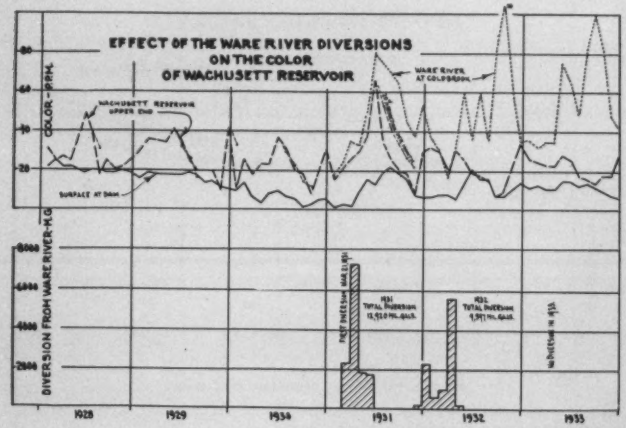

FIG. 24.

extent on record in this vicinity, but in the actual operation of the works with the water consumption so nearly equal to the safe yield, the short dry period of 1929 to 1931 was critical and by the middle of February, 1931 Wachusett was drawn to the low elevation of 45 ft. below the spillway.

Water Quality. While the District did not actually face a shortage of water, it did face a shortage of water of good quality, and the completion of the Ware River works made possible the replenishment of the supply at a time when it was most needed. Water was first diverted from the Ware River into Wachusett Reservoir on March 21, 1931, the storage in this reservoir having been depleted by some 44 billion gallons. Under these conditions, an excellent opportunity was afforded for studying the effect of the diversion of the water from the Ware River on the water in the reservoir. During the period from March 21, 1931, to the end of the year

12 920 900 000 gal. of water were diverted from the Ware River, and the color of this water varied from 33 to 80 p.p.m. The water of Wachusett Reservoir at the upper end, which was a mixture of water from its own tributaries with that discharged from the Ware River tunnel, had a color of 30 to 67 p.p.m. The effect of this diversion on the color of the water in Wachusett Reservoir is shown in Fig. 24.

A study of this diagram shows that the reduction of the color of the water in Wachusett Reservoir coincides with the curve referred to by Fair (using 20 per cent. per month reduction) in his discussion of a paper read by Kennison before this Association in 1928.*

JOHN L. HOWARD.† The Metropolitan Water Board acquired the supply works and pumping stations of the city of Boston January 1, 1898, and paid for them the sum of approximately $14 000 000. It also acquired Spot Pond from Malden, Medford and Melrose for the sum of $1 240 000. It constructed the Wachusett Dam, Aqueduct and Reservoir; completed the Sudbury Dam and Reservoir that was started by the city of Boston in 1894; laid about 165 miles of supply and distribution mains; built 4 pumping stations and an addition to the old high-service pumping station of the Boston Water Works; constructed the Weston Aqueduct and Reservoir; built two hydro-electric power stations and a 66 000-volt electric transmission line, 16 miles in length. Total construction expenditures amounted to about $48 000 000.

The value of sinking fund to retire bonds issued for construction purposes, prior to 1912, is approximately $32 000 000 and the net debt April 1, 1933 is about $12 500 000, excluding bonds issued for construction of the new works to the Ware and Swift rivers by the Metropolitan District Water Supply Commission. Since 1912 all bonds issued have been serial bonds, amounting to $4 300 000 and of these $1 300 000 have been paid. The storage reservoirs have a capacity of about 80 billion gallons. Four aqueducts have a total length of 56.5 miles. The Wachusett and Weston aqueducts have a capacity of 350 and 300 m.g.d. respectively, while the Sudbury Aqueduct has a capacity of 109 m.g.d. and the Cochituate Aqueduct 20 m.g.d. The distribution reservoirs have a capacity of about 2 400 m.g. and 3 standpipes a capacity of 4.83 m.g.

The operation of the hydro-electric stations over a period of 20 years produced a gross income of $1 300 000 and after deducting operating expenses and interest and sinking-fund requirement charges leaves a profit in 20 years of about $500 000, which has been used to reduce the assessments of the Metropolitan Water District.

The five pumping stations combined have a pumping capacity of 340 m.g.d. and a horse power of 7 500.

The Metropolitan Water District has an area of 174 sq. miles and an estimated population regularly supplied of 1 438 440 in 18 cities and towns.

*This JOURNAL, Vol. XLIII, No. 1.

†Deputy Chief Engineer, Metropolitan District Commission, Boston, Mass.

The average consumption in 1933 was 125 074 000 g.p.d., equivalent to 87 g.p.d. per capita.

There follows a summary of information on the old works of the Metropolitan Water District.

Wachusett Dam and Power Plant.

Wachusett Dam:

Work began in 1900 and was completed in 1906.

Length of main dam, including abutments	971 ft.
Length of waste weir	452 ft.
Elevation of top of main dam	El. 415 ft.
Height to top of dam above full reservoir level	20 ft.
Height to top of dam above finished surface of ground below the dam	114 ft.
Maximum height above rock foundation, about	207 ft.
Maximum thickness of dam at bottom	185 ft.
Thickness of dam at full reservoir level	25 ft.
Maximum depth of water at dam	129 ft.
Masonry required for dam	280 000 cu. yd.
Amount of contract	$1 603 635
Contractor	McArthur Bros. Co., of Chicago, Ill.
In charge of work	Winston Brothers & Locher

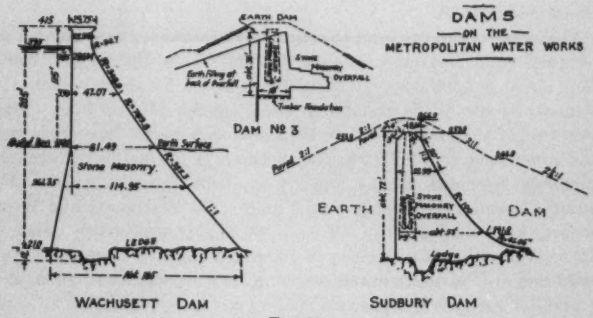

Fig. 25.

Power Plant:

Equipment installed in 1911.

S. Morgan Smith Co., of York, Pa., received contract for installing both hydraulic turbines and electric generators. Generators and other electrical equipment, furnished under a sub-contract by the Westinghouse Electric & Manufacturing Company of Pittsburgh, Pa.

4-48-in. C. I. pipes pass through the dam, inverts at El. 284 or 111 ft. below the high water in the reservoir.

Plant consists of 4-1 340 h.p. hydraulic turbine units of the spiral case, horizontal shaft type, each directly coupled to a 1 000 k.v.a., 60-cycle, 3-phase, 13 800 volt, alternating current generator, together with 1-90 h.p. and 1-110 g.p. horizontal shaft turbines,

each directly coupled to a 60-kilowatt direct current 125 volt generator used in exciting the main generators.

Speed of main generators is 400 r.p.m.
Diameter of penstocks ..... 48 in.
Diameter of wheels ..... 30 in.
Contract price of 4 complete units $71 500
Average head 97 to 98 ft.

2 lines of 3-conductor paper insulated lead-covered cables, 815 ft. long, composed of 19 strands of copper wires, tested to 30 000 volts, and laid in 3½-in. dia. Orangeburg fiber ducts.

Wachusett and Sudbury Reservoirs.

Wachusett Reservoir:

Work began in 1897 and was completed in 1905.
Reservoir first filled on May 10, 1908.
Area of watershed is 108.84 sq. miles.
Elevation of full reservoir above Boston City Base ..... 395 ft.
Water surface ..... 4 135 acres or 6.46 sq. miles
Land surface ..... 4 775 acres or 7.46 sq. miles
Full capacity ..... 64 968 000 000 gal.
Length ..... 8.41 miles
Maximum width ..... 2.05 miles
Maximum depth ..... 129 ft.
Average depth ..... 48 ft.
Length of shore not including islands ..... 37 miles
Soil removed from reservoir ..... 6 900 000 cu. yd.
Roads discontinued ..... 19.25 miles
New roads built ..... 11.8 miles
North Dike is about 2 miles long.
Maximum height above full reservoir ..... 17 ft.
Maximum width above full reservoir ..... 1 930 ft.
Amount of earth in North Dike ..... 5 161 814 cu. yd.
South Dike is about 2 800 ft. long.
Maximum height above full reservior ..... 19 ft.

Central Mass. R.R. — 6½ miles discontinued — 4½ miles new location — steel viaduct across the river is 917 ft. long with a maximum height above the river bed of 133 ft.

Sudbury Reservoir:

Work began in 1894 by the city of Boston and was completed by the Metropolitan Water Board in 1898.

Elevation of full reservoir above Boston City Base ..... 260 ft.
Water surface ..... 1 292 acres or 2.00 sq. miles
Full capacity ..... 7 254 000 000 gal.
Maximum depth ..... 65 ft.
Average depth ..... 19 ft.
Total length of shore including islands ..... 39 miles
Soil removed from reservoir ..... 4 593 000 cu. yd.
Roads discontinued ..... 8.42 miles
New roads built ..... 2.20 miles
Roads rebuilt ..... 5.24 miles
Area of watershed ..... 75.2 sq. miles

(See page 94 in 1898 report for quantities and cost.)

Wachusett Aqueduct.

First used in 1898.
Capacity ..... 362.6 m.g.d.
Total length ..... 12 miles
Rock Tunnel (12 ft. 2 in. wide, 10 ft. 10 in. high, lined) ..... 2 miles
Concrete and brick masonry covered with earth (11 ft. 6 in. wide, 10 ft. 6 in. high) includes Assabet River Bridge* ..... 7 miles
Open channel (20 ft. wide at bottom, sides 3:1) ..... 3 miles
Fall of 1:5000 in Tunnel from Station 1 + 41.85 to Station 100 + 00.
Fall of 1:2500 from Station 100 + 00 to Station 473 + 00.
Invert at Wachusett Dam ..... El. 284.52
Invert at Terminal Chamber ..... El. 267.63

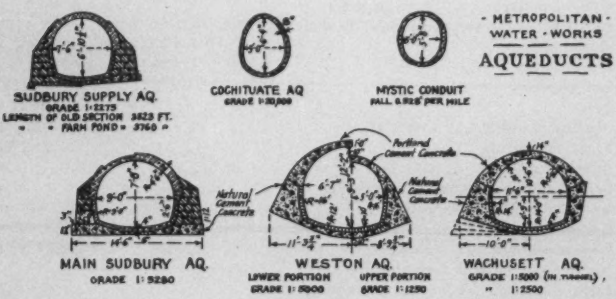

Fig. 26.

	Station.	Elevation of Invert.
Entrance of tunnel, Wachusett Dam	1 + 10.6	284.52
East end of tunnel	105 + 00	282.35
Gaging manhole	134 + 00	281.19
Assabet River Bridge	346 + 37.64	272.69
Terminal chamber at open channel	473 + 42.71	267.63
Intermediate Dam, open channel	520 + 00	264.60
Elevation of flashboards		272.6
Elevation of masonry crest		271.6
Lower Dam, open channel	632 + 25 ±	255.00
Elevation of flashboards		265.00
Elevation of masonry crest		264.00
Sudbury Reservoir at Lower Dam:		
Elevation of flashboards		260.00
Elevation of masonry crest		259.00
Elevation of bottom of reservoir		250.00

*Assabet River Bridge: Length (7 arches, 29 ft. 6 in. span) 359 ft. From surface of River to invert of Aqueduct, about 19 ft.

SUDBURY AQUEDUCT.

Built by city of Boston and taken by M. W. W. in 1898.

First used in February, 1878.

Capacity 109 m.g.d.

Framingham Dam No. 1 to Farm Pond Gate House, fall of 3.55 or 2.323 ft. a mile, size 7 ft. 6 in. wide at springing line × 6 ft. 10⅝ in. high for 1.5 miles.

Farm Pond Gate House to Chestnut Hill Reservoir, fall of 17.30 or 1 ft. a mile, size 9 ft. 0 in. wide at springing line × 7 ft. 8 in. high for 15.9 miles.

Total length ..... 91 850.1 ft. 17.38 miles

	Station.	Miles.	Elevation of Invert.
Framingham Dam No. 1	0		154.822
Farm Pond inlet chamber	41 + 67		152.990
Farm Pond Gate House (end of 7 ft. 6 in. × 6 ft. 10⅝ in. aqueduct)	79 + 18		151.269
Farm Pond Gate House (beginning 9 ft. 0 in. × 7 ft. 8 in. aqueduct)	1 + 00		151.352
Current-Meter Rating Station	59 + 00	1.1	150.216
Course Brook waste weir, Sherborn	112 + 00	2.1	149.212
Rockland St. Tunnel (beginning)	201 + 84	3.8	147.510
Bacon's waste-weir	271 + 40	5.2	146.193
Badger Hill Tunnel (beginning)	293 + 41	5.6	145.776
Waban Bridge (beginning)	381 + 97	7.3	144.079
Fuller's Brook waste-weir, Wellesley	459 + 00	8.7	142.620
West Siphon Chamber (beginning Rosemary Siphon)	540 + 00	10.2	141.086
East Siphon Chamber (end of Rosemary Siphon)	558 + 00	10.6	139.356
Charles River Bridge (beginning)	631 + 70	12.0	137.960
Sand Tunnel (beginning)	730 + 75	13.9	136.084
Clark's waste weir	738 + 15	14.0	135.944
Beacon Street Tunnel (beginning)	773 + 85	14.7	135.268
Terminal Chamber (end of aqueduct)	838 + 12	15.9	134.051

Waban Bridge, length (9 arches 44.67 ft. span) 536 ft., width 18 ft., height above bed of brook, 48 ft.

Charles River Bridge (Echo Bridge), length (7 arches) 475 ft., width 18 ft., height above bed of river, 79 ft., span of centre arch (radius 69.0 ft.), 129 ft.

Rosemary Siphon which crosses Rosemary Brook consists of 2 lines of 48 in., 1 line of 61 in. C. I. pipe, each line 1 800 ft. long.

There are 4 waste-weirs and 44 culverts on the aqueduct.

COCHITUATE AQUEDUCT.

Built by City of Boston, taken by M. W. W. in 1898.
First used in October, 1848.
Capacity 20 m.g.d.
Fall of 4.26 ft., or 3⅙ in. a mile, size 5 ft. wide at springing line × 6 ft. 4 in. high.
Length 13.7 miles.

	Station.	Miles.	Elevation of Invert.
Effluent Gate House			131.030
Dedman's Brook waste-weir	157 + 47	3.0	
Current Meter Rating Station, Wellesley	286 + 58	5.4	
Grantville waste-weir	350 + 59	6.6	
Beginning of 40 in. Siphon Pipe	411 + 18	7.8	
West Pipe Chamber (beginning Charles River Siphon)	412 + 64	7.8	129.040
East Pipe Chamber (end Charles River Siphon)	423 + 33	8.0	128.520
End of 40 in. siphon pipe	425 + 12	8.1	
Newton Center waste-weir	573 + 74	10.9	
Ward Hill Tunnel (beginning)	619 + 34	11.7	
Ventilator	646 + 80	12.3	
Connection to Influent Gate House, Chestnut Hill Reservoir	684 + 51	13.0	
Intermediate Gate House (west side)	698 + 35.3	13.2	127.17
Pipe connection from Sudbury Aqueduct Terminal Chamber	700 + 72.5	13.3	
Connecting Chamber, High Service Pump Station.	720 + 66	13.7	

Remarks: Charles River Siphon consists of 4 lines of C. I. Pipe, 2-30-in. and 1-36-in., 1 068.46 ft. long and 1-40-in. line 1 393.89 ft. long.

WESTON AQUEDUCT.

Built by Metropolitan Water Works.

First used in December 1903.

Capacity, 300 m.g.d.

Head House to siphon under N. H. R.R. at Nobscot: concrete aqueduct, fall of 14.48 ft. or 4.224 ft. a mile, size 10 ft. 0 in. wide at springing line × 9.2 ft. high for 3.43 miles. s = 1: 1250.

Siphon under N. H. R.R. at Nobscot: 130.2 ft. long, fall 2.92 ft.

Siphon at Nobscot to Channel Chamber: fall of 10.57 ft. or 1.056 ft. a mile, size 13.17 ft. wide at spring line, × 12.17 ft. high for 7.79 miles; s = 1:5 000.

Total length 13.42 miles.

	Station.	Miles.	Elevation of Invert.
Head House (end of pipes, entrance to Head House)	5 + 06		215.89
Tunnel No. 1 (beginning)	7 + 00		
Gaging Chamber No. 1	17 + 00	0.2	214.97
Tunnel No. 2 (beginning)	42 + 30	0.7	212.96
Tunnel No. 3 (beginning)	113 + 20	2.0	
Nobscot Siphon (beginning)	186 + 50	3.4	201.41
Nobscot Siphon (end)	187 + 80	3.5	198.49
Gaging Chamber No. 2	214 + 21	5.0	197.97
Siphon Chamber No. 1 (beginning of Sudbury River Siphon)	284 + 68	5.3	196.56
Siphon Chamber No. 2 (end of Sudbury River Siphon)	320 + 74	6.0	194.15
Siphon Chamber No. 3 (beginning of Happy Hollow Siphon)	353 + 53	6.6	193.46
Siphon Chamber No. 4 (end of Happy Hollow Siphon)	364 + 77	6.8	192.57
Tunnel No. 4 (beginning)	529 + 61	10.0	
End of Aqueduct (Channel Chamber)	597 + 38	11.3	187.92
Ash St. Bridge, East End	611 + 47 ±	11.6	187.50
Screen Chamber, West End	649 + 88	12.3	187.90
Terminal Chamber (end of aqueduct)	706 + 99	13.42	187.90
Entrance to Distributing Pipes	706 + 99.4		187.50

*Sudbury River Siphon steel pipe is 7.5 ft. in diameter and 3 605.55 ft. long.

*Happy Hollow Siphon steel pipe is 7.5 ft. in diameter and 1 124.02 ft. long.

Sudbury River Siphon crosses the river by arching the pipe to carry itself and the weight of the water; rise of pipe 5.5 ft.; length between abutments 80.0 ft.

Open Channel is 11⅓ miles from Sudbury Dam and is 1 400 ft. long, 12 ft. deep and 10 ft. wide at bottom, slope of sides 3:1.

*Provision for three lines of pipe, only one line laid.

SUDBURY POWER PLANT.

Regular service, 2 units installed, September 14, 1916.
Regular service, 3 units installed, November 27, 1916.
Hydraulic Machinery:
2–24 in. 325 h.p. vertical, shaft hydraulic turbines.
1–30 in, 1 000 h.p. vertical shaft hydraulic turbines.
Furnished by S. Morgan Smith Co., York, Pa.
30 in. on test at Holyoke, developed 87.4% efficiency.
24 in. on test at Holyoke, developed 85.3% efficiency.
Cost — $13 410.95, wiring, etc. $1 291.71.
Electrical Equipment, furnished by Westinghouse Electric & Mfg. Co., of Pittsburg, Pa.
1–800 k.v.a. alternating current generators.
2–250 k.v.a. alternating current generators.
At 360 r.p.m. to deliver 3-phase, 60-cycle, A.C.
At 2 400 volts, stepped up to 13 200 volts, and
2–750 k.v.a. transformers.
2–125 v.d.c. generators for exciters.
One exciter direct connected with electric motor, the other direct connected to top of shaft of 900 k.v.a. generator.
Cost of generators, $20 859.76.
Cost of lightning arrester chamber, $1 175.17.
3 Horizontal type hydraulic governors from Lombard Governor Co., $2 516.06.
1 of 6 000 ft. lb. capacity.
2 of 2 500 ft. lb. capacity.
Hand-operated travelling crane, 12 000 lb., cost $880.
2 Surge tanks for Weston Aqueduct 15 ft. × 7½ ft. × 7 ft. high.
1 Surge tank for Reservoir No. 3, 10 ft. × 6 ft. × 13 ft. high.
Cost of wheel pits and tanks, $46 906.14.
Overhead transmission line, 4 170 ft. long.
Underground cable 1 500 ft. paper-insulated lead-sheathed, 3 conductors, tested to 30 000 volts.
Total cost of whole equipment, $102 103.

DISTRIBUTING RESERVOIRS.

	High Water.	Available Capacity. Mil. Gals.	Area. Acres.	Earth Embankment.	Invert of Effluent Pipe.	Reservoir First Filled.	Maximum Depth. Feet.	Owned by.	Remarks.
Spot Pond Reservoir	163.00	1 791.7	308.7	166.00	137.80 N. 137.00 S.	1901	47	Met. W. W.	.....
Chestnut Hill { Lawrence Basin		168.6	37.7				17		
Chestnut Hill { Bradlee Basin	134.00	522.8	85.0	138.00	109.80	1868	32	Met. W. W.	1870
Weston Reservoir	200.00	200.0	60.0	205.00	187.90	....	27	Met. W. W.	....
Fells Reservoir	271.00	41.353	8.5	275.00	249.75	1899	21	Met. W. W.	....
Mystic Reservoir 560 x 350 ft.; 1½:1 slope	157.00	26.24	4.5	160.00	135.50	1863	22	Met. W. W.	....
Waban Hill Reservoir	264.50	13.50	2.8	270.00	246.00	1877	16.7	Met. W. W.	Bought Oct. 12, 1900
Forbes Hill Reservoir 150 x 350 ft.; 1¾:1 slope	192.00	5.116	1.226	195.00	174.50	1901	17	Met. W. W.	....
Bear Hill Reservoir	300.00	2.452	0.767	303.50	285.00	1902	15	Met. W. W.	....
Fisher Hill Reservoir 500 x 295 ft.	251.00	15.50	3.14	255.00	230.00	1887	18	Met. W. W.	1877 Bought 1913
Chelsea Reservoir 178 x 98 x 15 ft.	196.60	1.00	0.40	199.40	....	....	15	Chelsea	....
Revere Reservoir 150 x 156 x 18.6 ft.; 2:1 slope	194.50	1.818	0.54	195.10	.....	....	18.6	Revere	....

STORAGE RESERVOIRS AND DAMS.

	Drainage Area Above Outlet.	Area of Water Surface.	Capacity at High Water.	Maximum Depth of Water.	Length of Dam.	Length of Overfall.	Total Height of Dam.	Elevation Above Boston City Base.				Reservoir First Filled.
								Earth Embankment.	High Water.	Top of Flashboards.	Stone Crest.	
	Sq. Mi.	Acres.	Mil. Gals.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	
Wachusett	108.84	4 135	64 968.0	129	1 423	452	207	N. Dike 412 S. Dike 410-414	395.00	398.00	395.00	1908
Sudbury	22.28	1 292	7 253.5	65	1 940	300	..	266.00	260.00	260.16 259.67	259.00	1898
Framingham No. 1.	74.66	154	311.0	16	785.8	168.25	22	176.80	169.32	169.27 168.42	167.60	1879
Framingham No. 2.	45.14	134	562.6	20	1 340	184.38	26	183.83	177.87	177.12 176.49	175.87	1879
Framingham No. 3.	27.68	250	1 199.7	24	3 092	100.25	29	191.32	186.74	186.50	185.24	1878
Ashland	6.43	169	1 416.4	48	1 857	30	58	231.00	225.21	225.21 224.90	224.23	1886
Hopkinton	5.86	193	1 520.9	53	1 540	30	59	311.00	305.00	305.00 304.67	304.00	1895
Whitehall	4.35	601	1 256.9	18	..	..	13	339.90	337.91	Coffer Dam 339.91	Bottom of Gates 327.78	Natural
Lake Cochituate	17.58	730	2 097.1	70	125	63	..	148.36	144.36	144.40	142.05	Natural 1848
Farm Pond	0.54	165	167.5	12	1 115	..	12	164.00	159.25	.....	.....	....

STANDPIPES IN METROPOLITAN DISTRICT.

Location.	Dimensions.		Capacity Brim Full Gallons.	Gallons per Foot.	Elevation at Top.	Elevation at Bottom.	Owned by.	Date Erected.
	Diameter.	Height.						
Arlington	75 ft.	61 ft. 3 in.	2 000 000	33 045	443.25	382	Met. W. W.	1921–22
Bellevue	100 ft.	44 ft. 3 in.	2 500 000	58 700	376.25	332.00	Met. W. W.	1914–15
Forbes Hill	30 ft.	64.33 ft.	330 000	5 287	251.00	188.5	Met. W. W.	1900
Lexington	30 ft.	105 ft.	550 000	5 287	443.43	.....	Lexington	1912
Nahant	30 ft.	95 ft.	502 300	5 287	170.0	75.0	Nahant	1902
Quincy	35 ft.	60 ft.	425 000	7 200	249.0	189.0	Quincy	.....
Swampscott	40 ft.	65 ft.	611 000	9 400	200.0	.....	Swampscott	.....
Wakefield	40 ft.	62.3 ft.	585 600	9 400	296.0	.....	Wakefield	1883
Winthrop	40 ft.	100 ft.	940 000	9 400	204.0	.....	Winthrop	1910
Orient Heights	24 ft.	40 ft.	133 364	3 384	204.4	.....	Boston	1889

Pumping Engines.

Station.	Eng. No.	Diam. of Steam Cylinders.	Diam. Plunger	Stroke.	R.P.M.	Cu. Ft. per Rev.	M.G. 24 H.	Speed Ft.Min	H.P.	Head Ft.	Suct. In.	Dis. In.	Erected	Cost.	Designer.	Builder.
Chestnut Hill No. 1	3	13.7–24⅜–39	17.5	48	50	39.91	20	400	550	137.5	48	36	1895	140 000	E. D. Leavitt	Quintard Iron Works
Chestnut Hill No. 1	4	30–56–87	42	66	17.5	158.75	30	192.5	800	140	48	48	1898	139 562	E. P. Allis Co.	
Chestnut Hill No. 1	16	23–54	23.75	36	38.9	35.25	15	203.4	448	133	36	30	1922	75 900	Worthington St. Pump Corp.	
Chestnut Hill No. 1	19	a.			525	8.845	50	....	1 390	135	42	36	1932	41 816	Warren St. Pump Co. and	
Chestnut Hill No. 1	20	b.			900	1.547	15	....	623	200	24	24	1932	22 385	Westinghouse Elec. & Mfg. Co.	
Chestnut Hill No. 2	5–6–7	17–31¼–48	37	60	29	112	35	290	960	45	36	36	1899	45,500 ea.	Holly Manufacturing Co.	
Chestnut Hill No. 2	12	32–60–90	43.5	60	24	154.81	40	240	1 036	132	36	36	1911	96 769	Holly Manufacturing Co.	
Arlington	10	14–28	9.5	18	63	2.78	1.89	186	76	290	12	12	1907	6 370	Allis Chalmers Co.	
Arlington	15	c.			1675	0.165	3	....	171	320	12	10	1918	9 700	F. A. Mazzur | Allis Moore Co.	
Arlington	18	16–32	10.375	30	51	5.45	3	255	178	305	16	16	1928	26 119	Murray Iron Works	
Spot Pond	8	21–42	14⅞–21	48	51	19.24	10.57	204	230	125	36	30	1900	.....	Taken from City of Boston	
Spot Pond	9	22–41½–62	30.5	60	24.4	76.17	20	244	463	125	30.5	36	1900	62 500	Holly Manufacturing Co.	
Spot Pond	17	24–54	26.5	36	41.6	45.30	20	249.6	479	125	42	36	1925	6 900	Worthington St. Pump Corp.	
Hyde Park	13–14	13–27	11.5	24	50	5.58	3	200	85	140	20	16	1913	8 190 ea.	Laidlaw Dunn Gordon Co.	
Pegan	1–2	5–10	d		300	1.238	4	....	25	11	8 10	8 10	1908	750 ea.	Fore River Ship and Engine Co.	

a.–b. 19–20 Warren Centrifugal Pumps driven by Westinghouse steam turbines.
c. 15 Two 8-in. Allis Chalmers Centrifugal Pumps in series driven by Moore steam turbine.
d. 1–2 8 and 10 in. Centrifugal Pumps driven by Marine Engines.

BOILERS.

Station.	No.	Type.	Shell.				Tubes.			Heating Surface		Steam Nozzles	Grate.	Builder.	Erected.
			Diam. In.	Length Ft. In.	*Pressure.	Thickness. In.	No.	Diam. In.	Length. Ft. In.	Water. Sq. Ft.	Total. Sq. Ft.	Diam. In.	Area. Sq. Ft.		
Chestnut Hill No. 1	20–21	Ver.	98	24 0¾	189	15/16	400	2	15 0	2 238	3 028	5	42.12	Dillon Steam Boiler Works	1921
Chestnut Hill No. 1	22	Ver.	98	24 0¾	189	15/16	400	2	15 0	2 238	3 028	5	42.12	Coatesville Boiler Works	1922
Chestnut Hill No. 1	26–27	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	International Eng. Co.	1931
Chestnut Hill No. 1	28	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1932
Chestnut Hill No. 2	15–16	Ver.	109⅞	24 1½	166	⅞	484	2	15 0	2 704	3 638	5	54.54	Robb Mumford Boiler Co.	1911
Chestnut Hill No. 2	29–30	Ver.	98	24 1½	166	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1932
Chestnut Hill No. 2	31	Ver.	98	24 1½	166	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1932
Arlington	13–14	Hor.	54	17 5½	160	7/16	60	3	17 0	873	873	4	22	Hodge Boiler Works	1907
Arlington	19	Hor.	54	17 5½	160	13/32	60	3	17 0	873	873	4	22	New England Iron Works	1918
Spot Pond	23	Ver.	98	24 0¾	189	15/16	400	2	15 0	2 238	3 028	5	42.12	Coatesville Boiler Works	1923
Spot Pond	24	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	Dillon Steam Boiler Works	1930
Spot Pond	25	Ver.	98	24 0¾	189	15/16	392	2	15 0	2 207	2 990	5	42.12	International Eng. Co.	1931
Hyde Park	17–18	Hor.	54	14 5⅞	150	7/16	62	3	14 0	748	748	4	20	Robb Engineering Co.	1912
Pegan	1–2	Ver.	54	11 0	150	7/16	148	2	7 5	391	573	2.5	12.56	Dillon Steam Boiler Works	1923

*Pressure, pound per square inch.

DUTIES OF PUMPING ENGINES.

Station.	Engine No.		Date of Test.
Chestnut Hill No. 1	3	150 million ft. lb. per 100 lb. of coal	1895
Chestnut Hill No. 1	4	176 million ft. lb. per 1 000 lb. of steam	1900
Chestnut Hill No. 1	16	148 million ft. lb. per 1 000 lb. of steam	1922
Chestnut Hill No. 1	19	160 million ft. lb. per 1 000 lb. of steam	1933
Chestnut Hill No. 1	20	144 million ft. lb. per 1 000 lb. of steam	1933
Chestnut Hill No. 2	5–6–7	159 million ft. lb. per 1 000 lb. of steam	1901
Chestnut Hill No. 2	12	175 million ft. lb. per 1 000 lb. of steam	1912
Arlington	10	116 million ft. lb. per 1 000 lb. of steam	1909
Arlington	15	90 million ft. lb. per 1 000 lb. of steam	1918
Arlington	18	142 million ft. lb. per 1 000 lb. of steam	1928
Spot Pond	8	127 million ft. lb. per 100 lb. of coal as No. 4 at Mystic	1895
Spot Pond	9	172 million ft. lb. per 1 000 lb. of steam	1901
Spot Pond	17	142 million ft. lb. per 1 000 lb. of steam	1925
Hyde Park	13	120 million ft. lb. per 1 000 lb. of steam	1913
Hyde Park	14	121 million ft. lb. per 1 000 lb. of steam	1913

PROCEEDINGS.

MARCH MEETING.

HOTEL KENMORE, BOSTON, MASS.

Wednesday, March 21, 1934.

President E. Sherman Chase in the Chair.

Secretary GIFFORD. At the Executive Committee meeting this morning the following were elected members: Fred O. Walker, Superintendent, Rumford, Me.; Ralph Adams, Chairman, Board of Public Works, Arlington, Mass.; George B. Mebus, Designing Engineer, Glenside, Pa.

The PRESIDENT. I regret to announce the death on Sunday of Mr. Charles E. Pratt of the Chapman Valve Company. Mr. Pratt was known to most of us as an active participant in these meetings for a great many years. Will the members please rise for a moment in respect of the memory of Mr. Pratt.

(All the members rise and stand in silence.)

The Report of the Committee on Hydrant Markings was read by Mr. Harry U. Fuller, Chairman. (See p. 252.)

On motion, duly seconded, it was VOTED that the report of Committee on Hydrant Markings be accepted and placed on file.

A series of motion pictures illustrating Civil Works Administration water-works construction in Leominster, Mass., was shown and explained by W. Guy Classon, Superintendent and Registrar, Leominster Water Works.

A discussion on Cold Weather Troubles was opened by Percy A. Shaw of Manchester, New Hampshire, and participated in by Messrs. George E. Hawkins, Harry U. Fuller, David A. Heffernan, W. Guy Classon, Henry T. Gidley, Frank J. Gifford, George McKay, Hervey A. Hanscom, Edmund C. Sanderson, William F. Sullivan, Samuel A. Agnew, Henry E. Halpin, Charles W. Mowry, Roger W. Esty, Albert A. Ross, and Harold L. Brigham.

(*Adjourned*)

REPORT OF COMMITTEE ON HYDRANT MARKINGS.

To the New England Water Works Association:

Your committee on the Standard Markings of Fire Hydrants submits the following report:

This subject is of importance to all water utilities and fire departments. If every hydrant in a water works is tested to learn the amount of water it will deliver at a given residual pressure and the hydrant is marked to show the result, the fire department will be able to work to better advantage and the water-works officers will have valuable records. If the system of marking is uniform, fire departments from other towns will be able in time of emergency to locate at the proper hydrants. It is desirable that the capacity of all hydrants be determined at the same residual pressure and that the full capacity when supplying pumpers be known. For this reason a residual pressure of 10 lb. per sq. in. has been adopted for the standard.

Markings. Markings should be determined by flow test, and should be divided into three classes, A, B, and C.

Class A. Hydrants which have sufficient capacity to supply 1 000 or more gallons per minute.

Class B. Hydrants which have sufficient capacity to supply not less than 500 gallons per minute.

Class C. Hydrants with insufficient capacity to supply 500 gallons per minute.

Post Hydrant Markings. Barrels of all hydrants should be painted chrome yellow for visibility.

Nozzle and Cap Markings:

Class A. Tops and nozzle caps should be painted green.

Class B. Tops and nozzle caps should be painted yellow.

Class C. Tops and nozzle caps should be painted red.

The colorings conform to standard traffic signals with green the most desirable, and red the least desirable.

Flush Hydrant Colorings. All location markers for flush hydrants should carry upon their background, the colors designating the class in which that particular hydrant falls.

All flush hydrant covers should carry upon the under side, preferably within the circle, the proper color as a background upon which any necessary data could be stencilled in black.

Respectfully submitted,

(Signed) HARRY U. FULLER,
GEORGE H. FINNERAN,
FRANK E. WATERMAN.

Remarks by Mr. Harry U. Fuller.

Some question has been raised as to the amount of information that it is proper to attempt to convey by marking hydrants. Of course it is desirable to convey all the information possible, but it is not desirable to convey misinformation. What should be done is controlled by the degree of accuracy of the information conveyed. If the hydrants are marked as suggested in the report the information is not perfectly accurate because the number of nearby hydrants being used affects the results.

It has been suggested that hydrants be tested in groups instead of singly, but if that were done there is no standard of how many to use or which ones to use. One of the advantages of testing the hydrants singly is that the tests can be repeated at intervals and the effect of improvements to the system can be learned as well as reduced capacity due to the age of the pipes.

It has been suggested that the size of the main be painted on the hydrant, but if the hydrant were on a 6-in. main the supply might be one fire stream or six, depending on the length of 6-in. supply. That information would be of no help.

Another suggestion has been made that more colors or more figures be used to represent smaller differences in quantity. I should call that an unreasonable attempt at refinement and not justified. It seems to me that the suggested report is as far as we should go at this time. In those places where I have known of the suggested marking being applied it has been well received by the fire departments and they consider the idea good.

When the capacity of the hydrant is greater than 2 000 gallons per minute, a number should be painted on the barrel of the hydrant signifying the number of thousand gallons per minute exceeded.

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The cuts shown are our two and three piece sliding type adjustable valve boxes.

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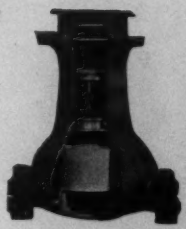
Waterworks men who installed Trident and Lambert Meters a generation ago would be amazed to see them in service today. Not because they last . . . but because inside the old meter casing they would find new interchangeable improved parts, bringing the old meters up-to-date, extending profitable operating life indefinitely! This guarantee of interchangeability covers the Trident or Lambert meters we sell you TODAY. For years we have pioneered in meter progress. We are doing it today. We'll do it tomorrow. Any future improvements in Trident or Lambert Water Meters will be interchangeable with present parts. Your meters will be a lasting investment. A small stock of interchangeable parts will economically keep your installation in perfect operating condition. A type of meter for every condition of service. Write for Catalogs to the Neptune Meter Co. (Thomson Meter Corp.), 50 West 50th St., (Rockfeller Center), New York City . . . . . or . . . . . Neptune-National Meters, Ltd., Toronto, Ont., Canada.



The famous Trident Frost Proof Meter. Its parts are interchangeable. Among other Trident features it embodies Oil-Enclosed Gear Train, Anti-Friction Disc Thrust Roller, Snap Joint Disc Chamber; special Heat proofed and Removable bushings on Gear Train and Register.

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In EASTON, *Penna.*

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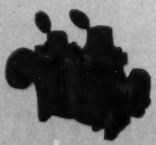

The Lehigh Water Company, at Easton, Penna., has recently completed a modernization program which makes their plant and distribution system rank as one of the finest in the country for the size of the community which it serves. The most modern equipment was installed and the company may well be proud of their system and the high type of service which it renders.

As a part of this modernization program, Pittsburgh Water Meters were installed. The system is now 100% metered. Universal metering has proven beneficial to both the Lehigh Water Company, in that they receive an equitable return on their investment, and to their customers, who are billed only for the actual amount of water used.

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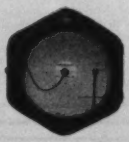
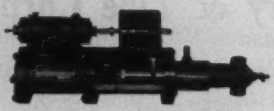
THE HERSCHEL STANDARD VENTURI METER TUBE AND TYPE M REGISTER-INDICATOR-RECORDER for Main Pipe lines.



THE CHRONOFLO ELECTRIC FLUID METER for long distance transmission of flow rates, reservoir levels and elevated tank pressures.



THE TRET-O-UNIT MECHANICAL PROPORTIONER for automatically feeding chemical solutions to water lines under pressure.

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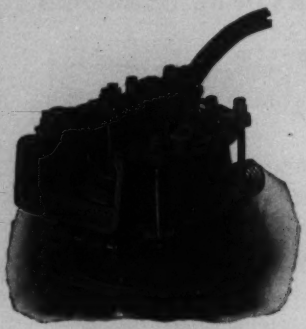

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INCORPORATED 1868

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Our experience in building meters extends over a period of more than sixty years. The experience of some of our customers in *using* meters of our manufacture extends over an almost equal period of time.

National Meters have always been known as accurate, dependable, economical meters. They are better today than ever before.

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The NASH Disc-type Meter

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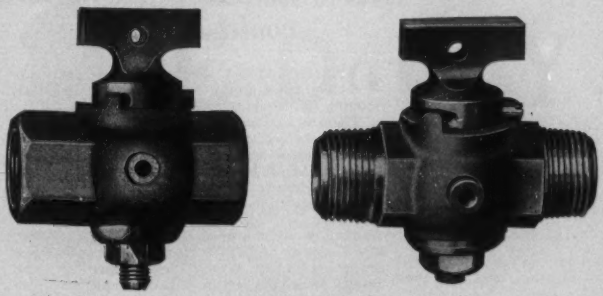
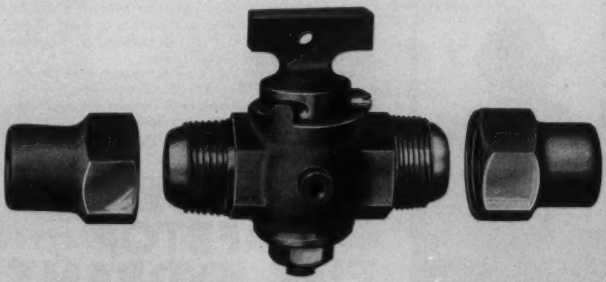
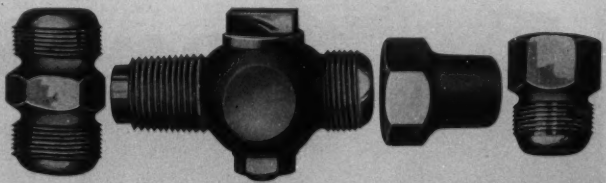
Full Size Waterway—Easy Turning—Tested at 200 lbs.



TEE HEAD NEWPORT STOP AND WASTE COCKS



TEE HEAD NEWPORT STOP AND WASTE COCK
With Double Copper Tubing Connections



Cocks for ¾" Copper Tubing have 1" I. P. Male Threads and Cocks for 1" Copper Tubing have 1¼" I. P. Male Threads. This allows our cocks to be used with regular I.P. fittings when necessary.

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which no other fire hydrant could undergo

DELIBERATELY driving a heavy automobile truck into a fire hydrant seems like looking for certain trouble.

There is only one hydrant for which this test has ever been offered as convincing proof that it will not cause a serious accident and expensive repairs if struck by a smashing impact.

The Kennedy SAFETOP Fire Hydrant has been subjected to this test of an actual collision; not once, but dozens of times. Each test has uniformly demonstrated the same results: that the Kennedy SAFETOP Hydrant, if struck by a smashing impact, breaks cleanly apart at the Safety Breakable Section without damage to the major and expensive parts of the hydrant; that no flooding or serious personal injuries result from the collision; and that the hydrant can be repaired and put back into service by one man, within half an hour, without excavating, and at a total cost of less than $10.00.

Be sure to write for full information and prices on the SAFETOP Fire Hydrant—*the only hydrant that has actually undergone collision tests to prove its exclusive advantages.*

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More Than Sixty Years of Experience Goes Into All

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BUILT TO MEET EVERY EXACTING REQUIREMENT OF WATER-WORKS SERVICE

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Stems cannot be bent in closing the hydrant.
Nothing can lodge on the valve seat to prevent closing of the valve —
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The position of the stem of the hydrant indicates accurately the position of the valve —
Whatever can pass the valve opening will pass the standpipe —
No frost case necessary —
All of these points are explained thoroughly in our catalogue, which will pay you to study.

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Has its distinctive and individual features. The double disk gates are free to adjust themselves in different positions every time the valve is closed, thereby keeping smooth and tight faces. And two bronze hooks on the ball loosely engage with the gates to prevent them from spreading at the top. The center bearing gates are forced to their seats with equal pressure at all points. There is a good deal more worth knowing about Eddy Valves. Furthor details will be sent for the asking.

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The Eddy line of water-works products also includes horisontal and vertical check valves, foot valves, hydraulically operated valves, indicator posts, etc.
All goods made by the Eddy Valve Company are manufactured exclusively at Waterford, New York.

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For calking pipe joints under the most difficult conditions.

For overhead joints, or in wet places where the use of molten lead is not only impracticable but dangerous — **LEAD WOOL** may be used to advantage.

It makes an absolutely tight joint which will withstand the highest pressure, yet be sufficiently elastic to allow considerable sagging or settling of the pipe without danger of a leak.

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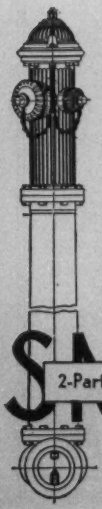

Minimum Trouble and Expense When Breaks Come

THE Smith Two-Part Standpipe Hydrant was designed to make the expense as small as possible when there is a break as the result, for instance, of careless auto truck driving.

The upper section will always break first. It can easily be renewed. And of course there can be no leakage of water because the main valve is of the compression type.

THE A. P. SMITH MFG. COMPANY
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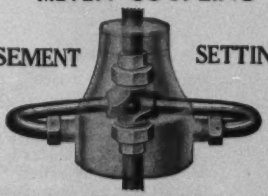
The
COPPERHORN
METER COUPLING
for BASEMENT SETTINGS



- 1 Meter set horizontally at desired height in vertical pipe without ells, nipples or couplings.
- 2 All copper water way. No rusting or corrosion. Freezing will not burst it.
- 3 Smooth oversize tubing and sweeping bends, maximum flow.
- 4 Meter changes do not disturb piping whether the meter is in or out.
- 5 On new services it serves as a spacer to connect in the piping.
- 6 Provides a permanent electrical ground connection.

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The FORD METER BOX COMPANY
Wabash, Indiana

The Hays Copper Plumbing Method

Regular Corporation Stop

Adapter

In making an installation with Hays Copper Service Fittings the regular Corporation Stops can be used in conjunction with the adapter. It is not necessary to procure new mandrels, as the Corporation Stops are inserted with the regular mandrels, in the usual manner, after which the adapter is attached.

Open View of Adapter

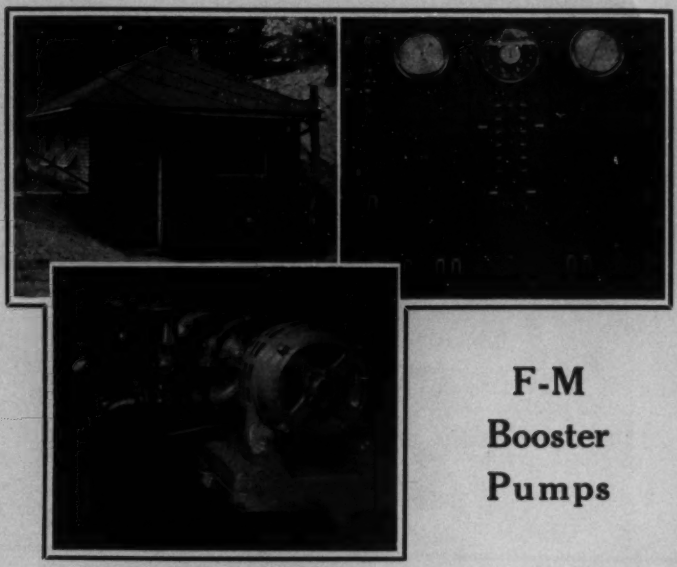
These adapters are made with Female Thread for Increased I. P., Water Works Standard or Lead Flange.

To use standard Corporation Stops with Increased I. P. or Water Works Standard Thread, and where a direct connection of the Corporation Stop and Copper Pipe is desired, the above stop has been designed. (See cut above for sectional view of joint.) These stops permit a Hays 90° seal with the use of regular male mandrels.

Write for complete details of

The Hays Copper Plumbing Method for Water Works and House Plumbing

Hays Manufacturing Co. Erie, Pa.



F-M
Booster
Pumps

At SOUTH HADLEY, MASS.

Shown above are the exterior of the South Hadley Water Works Pumping Station, the two F-M Motor Driven Centrifugal Pumps installed in the station and the control equipment. The station is located at one extreme end of the town and takes water from a small reservoir under a positive head of approximately 12 or 15 feet and pumps it through the main section of the town to a standpipe at the other end. Both pumps and motors are of ball bearing design, which construction was selected because of the desirable feature of lubrication. The bearings require attention only once a year.

FAIRBANKS, MORSE & CO.

178 Atlantic Avenue, Boston, Mass.

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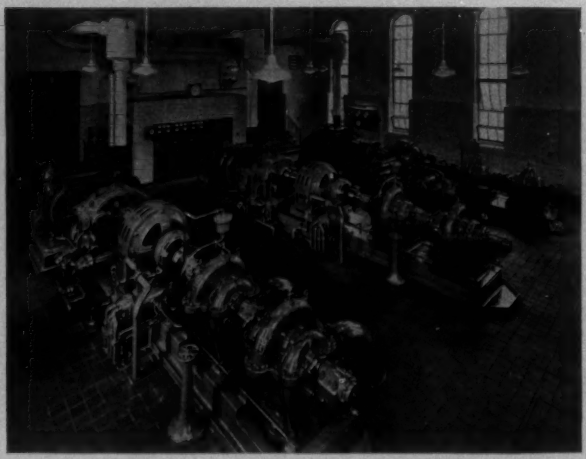
PUMPS · DIESEL ENGINES · MOTORS

DE LAVAL

Geared Turbine Driven

PUMPS and GENERATORS

The Most Modern and Efficient Pumping Station in New England



CITY OF NEWTON, MASSACHUSETTS

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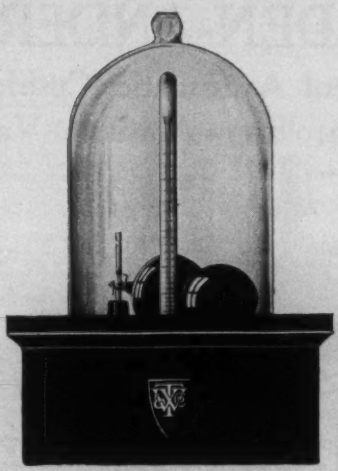

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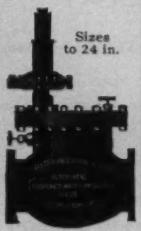

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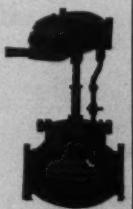

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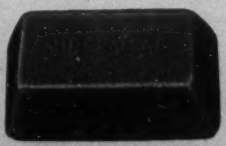

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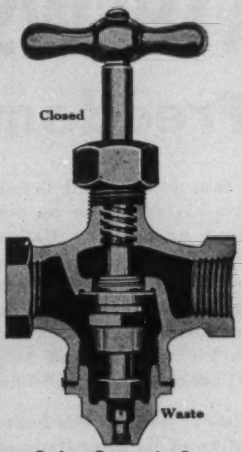

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